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ANALYSIS AND DESIGN OF FLEXIBLE UNDERGROUND STRUCTURES

by

N. M. Newmark, J. W. Briscoe, and J. L. Merritt

Prepared for

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
CORPS OF ENGINEERS
VICKSBURG, MISSISSIPPI

Contract No. DA-22-079-eng-225

Final Report

First Phase

31 October 1962

NATHAN M. NEWMARK
111 Talbot Laboratory
Urbana, Illinois

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of slope (the intersection of the slope with the original ground) to the nearest point on the structure is at least four times the height of the mound. Even where a greater depth of cover is provided, a slope no steeper than 1 horizontal to 2 vertical is permissible within the boundary of Region A as shown in Fig. 4.1. Region A in Fig. 4.1 is specified to allow for greater depths of

(Following to be inserted at bottom of p. 60.)

4.3.2 Completely Buried Barrel Arches. A completely buried barrel arch develops its resistance as a hoop compression. The yield thrust S developed under this condition is the product of the yield compressive stress σ_y and the area of the arch rib cross-section A.

$$S = \sigma_y A = r\, br' \text{ or } r = \frac{\sigma_y A}{br'}$$

where r = yield resistance measured in the same terms as the peak vertical stress acting at the crown of the arch,

b = width of the section of arch considered,

r' = mean radius of the arch.

Empirical results summarized in Appendix F, indi-

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ABSTRACT

The original purpose of this investigation was to provide a protective shelter that could be constructed and occupied in a forward area within one week's time by an engineer platoon with a normal strength (at the time this study was initiated) of 51 men. However, it should be pointed out that the structure as finally designed can be built and occupied by any troop unit of comparable strength with a minimum of engineer equipment, assistance and engineering supervision.

While the original concept was to design a forward area troop shelter, a subsequent modification called for consideration of the proposed troop shelter as a civil defense shelter. The basic shelter can be used for civil defense, although it was not designed specifically for that purpose. Therefore, some minor modifications of the basic shelter would be necessary before it would serve satisfactorily as a civil shelter.

Because of the time limitations and adverse conditions which generally exist in a forward area, the structure designed herein is made as simple as possible. Little cast-in-place concrete is required in its construction, and no special equipment or skill are required for its erection. Despite these limitations the standard design presented herein will successfully withstand an overpressure of 100 psi at the surface above the shelter, and the nuclear effects associated with this overpressure, from a bomb even in the high megaton range of yield. With relatively minor modifications of a standard structure, protection can be provided from an overpressure, and associated effects, of as much as 300 psi.

The basic (51-man) shelter is 16 by 48 ft. in plan. It consists of steel arch ribs, 8 ft. in radius, which support timber blocks. The arch ribs, made from inverted structural tees, are supported on timber sills. End bulkheads consist of vertical wide flange posts which support timber blocks spanning horizontally between the posts. The top of the posts are supported by the main structure itself, while the bottom of the posts are supported by a horizontal truss which is, in turn, supported by the ends of the sills.

The entrance structure consists of two principal units; the vertical entranceway and the horizontal passageway. The vertical entranceway is composed of two concentric corrugated pipes. The space between the two pipes is filled with concrete in order to make the overall structure act as one unit. The horizontal passageway consists of prefabricated steel frames with timber and steel plates spanning between the frames. The passageway is of modular construction and serves the additional function of providing space for such items as mechanical equipment, CBR filters, and decontamination equipment if desired.

Suspended bunks for 51 men have been provided, with the bunks easily retractable when not in use to provide recreation or work area. Sufficient storage is provided for supplies for a one-week occupancy by 51 men, and additional space can easily be provided for in the plan. The basic report deals only with the standard 51-man structure, designed for a 100 psi overpressure, with a vertical personnel entranceway. Alternative designs, such as other possible entranceways, side by side spacing and other modifications are included in appendices.

The structure is designed so that it can be provided in a series of kits. Each kit contains all parts necessary for the erection of a complete section or unit of the structure. Use of these kits provides almost complete flexibility in the size and arrangement of the structure. For instance, the shelter may be of any multiple of 12 ft. in length by specifying fewer or more main shelter kits. Similarly, additional ventilation or entrance structures may be specified by adding additional passageway or ventilation kits.

Detailed fabrication and erection plans are included in the appendix.

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ACKNOWLEDGMENT

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Studies made for this report were prepared under the general supervision of Dr. Newmark, and under the immediate direction of John W. Briscoe. Personnel contributing to this report were John D. Haltiwanger, James E. Stallmeyer, C. W. Lewitt, and J. T. Hanley, and to a major degree Allen F. Dill. J. L. Merritt and J. W. Briscoe are responsible for the major part of this report.

1. INTRODUCTION

1.1 OBJECT AND SCOPE

This report summarizes the results of investigations made in the design of a shelter for rapid and economical construction by engineer troops in the field. Because of this special emphasis, it was necessary to devise a shelter which could be erected rapidly with the normal or standard equipment available to an army engineer platoon. While these limitations of time and equipment presented some unusual problems, it appears that a satisfactory structure can be erected and occupied within a period of one week.

For the reasons discussed in Sections 1.2 and 3.1 only buried construction above the water table and bedrock was considered in this study. The type of structure that appears most feasible is a completely buried (including earth-mounded) arch consisting of steel ribs supporting timber, metal or precast concrete elements between the ribs. In this report timber elements only are emphasized, since this type of structure appears most feasible for rapid construction in a forward area. Vertical ladder-type entranceways only are considered in the basic report. Alternate entranceways, both personnel and vehicular are presented but not designed in the appendices. Also included in the appendices are studies of the spacing of adjacent basic structures.

A brief summary of the results of the study to determine the feasibility of various structural types are presented in this report. Design criteria for these structural types also are given. Design calculations for the proposed troop shelter are included in an appendix. Also included in the appendix are the studies which have been made to determine

the applicability of the design criteria for arches presented in the basic report. The results of these studies indicate that the design procedures currently given are safe as well as simple.

1.2 HISTORICAL BACKGROUND OF UNDERGROUND STRUCTURAL DESIGN

It long has been recognized that buried construction is highly advantageous for providing protection from shock loads and fragments resulting from explosions of conventional shells and bombs nearby. Also it has been recognized almost since the advent of nuclear weapons that buried shelters are desirable for protection against the radiation and blast effects of nuclear bursts. The decision between supplying protection by buried structures or by surface structures primarily is based upon the relative costs of the two configurations. However, it generally may be said that buried construction becomes economical for protection against the effects associated with a side-on overpressure of 30 psi or more. This economy results from the very large antisymmetrical loads (reflected and dynamic pressures) which develop at ranges from the burst corresponding to overpressures in excess of this value and the consequently larger structural strengths required as compared to strengths of buried structures at the same range. Furthermore, it usually is more economical to provide protection against prompt and residual radiation by covering the structure with earth than to gain this protection by adding large thicknesses of other structural materials to the exposed structure to accomplish this protection.

Significant developments in the design of underground protective structures were made during World War II. However, because of the relatively

concentrated forces associated with the high explosive weapons of that period, primary consideration was given to the effects of contact explosion or very near misses in the design of these early structures. As a result of these considerations protective structures were designed to resist penetration of the weapon itself, penetration of fragments from the weapon, and also the forces generated by a weapon exploding within a few feet of the structure. To provide protection against penetration of the weapon and fragments from a weapon very massive structural elements generally were required. Damage by shock from a near miss explosion acting on a structure proportioned on the basis of penetration resistance generally required a detonation so close that the structure was within the rupture zone of the crater formed by the weapon.

The advent of nuclear weapons required a re-evaluation of the design criteria then in use for buried protective structures. An obvious criterion which could no longer be applied was the penetration of the weapon itself. Thus, the only remaining criterion from the classical problem was that of resisting the shock induced by the detonation of the weapon at some distance from the structure. Also an additional problem, that of protecting against radiation, presented itself. Since a few feet of earth generally will provide protection against nuclear radiation, this last problem can be resolved fairly easily. However, there were many problems to be considered regarding the shock produced by a nuclear weapon.

It may be shown, Ref. (1.1),* that the attenuation with distance of the ground shock from a weapon detonated below ground <u>in soil</u> is much more rapid than is the attenuation with distance of the air blast from the

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*Numbers in parentheses throughout this report refer to corresponding entry in Appendix A, References.

same weapon detonated at or near the surface. Thus it may be concluded that, except at distances closer than approximately the zone of rupture of the crater, the effects of a surface burst of a nuclear weapon are more severe than those of an underground burst in soil of the same weapon. Criteria for the design of structures to resist these effects are presented in Refs. (1.2 and 1.3 for example). The criteria given herein are modified from these previously derived criteria to make them directly applicable to the type of construction considered in this report.

1.3 NOTATION

A detailed listing of the notation used in this report is included in Appendix B.

2. SUMMARY OF DESIGN CONSIDERATIONS

In this chapter the several factors to be considered in the design of a shelter for an army engineer platoon are summarized. Some of these requirements are self-evident; therefore, they are not discussed. Other requirements are based on rather detailed study, the results of which are summarized in the following chapters.

2.1 FUNCTIONAL AND ARCHITECTURAL FACTORS

2.1.1 Size of Structure Required. An engineer platoon normally consists of 51 men. The minimum requirement specified by WES stipulated that the structure must provide for housing and maintaining this number of men for a period of one week. Because of the radiation hazard outside, these men must remain constantly in the structure for approximately this period of time. The structure must provide sufficient space for sleeping and recreation. Also a one week's supply of food, water and other provisions must be stored within the structure. Sanitary facilities, communications equipment and possibly power generating devices must also be housed. Use of suspended retractable bunks in three decks allows the main shelter area to be used both for sleeping and recreation. Equipment and storage of provisions in addition to the sleeping and recreational requirements define a minimum gross floor area of 560 sq. ft. When all criteria and their effects upon structural integrity and costs are considered it appears the best structural type for the main structure excluding the passageway will provide a gross floor area of 688 sq. ft. or an average of 13.5 sq. ft. per man. A lesser area in the main structure can be provided by using the passageway for storage of provisions and equipment.

2.1.2 Structural Type Required. Arch-shaped ribs supporting timber, corrugated metal or precast concrete elements are suited best for structures located above the water table which must be constructed rapidly with possibly unskilled labor. Because of the many difficulties associated with construction below the water table, such construction should be avoided wherever possible. It is imperative that the structure be completely buried in any case. This burial may be accomplished by locating the structure entirely below the natural surface of the ground or by covering the structure with extensive fill.

Because the clear span required in the structures being considered can be limited to less than 10 ft., rectangular structures could be used instead of arch spans. However, studies summarized herein indicate arched construction is preferable.

2.1.3 Requirements for Radiation Protection. To meet the limitations imposed by construction techniques, it is feasible to provide protection against radiation by specifying sufficient earth around the structure to reduce radiation levels within the structure to a tolerable range. The study reported in Appendix D indicates that the specified minimum depth of cover over the crown of 4 ft. appears to be sufficient to protect against radiation of the greatest intensity to be expected at a range corresponding to the 100 psi overpressure from a near-surface detonation. Therefore, the structure may be proportioned to resist only the shock loads when this minimum cover is provided.

2.1.4 Terrain Factors. For purposes of concealment it is imperative to blend the fill over the structure so that it matches as closely as possible the natural surroundings. Generally cut-and-cover construction

will be used. However, where there is sufficient topographic relief and sufficient time to tunnel horizontally into a hillside, this means of construction might be considered.

2.1.5 Location of Water Table and Bedrock. As mentioned above it is exceedingly desirable to avoid construction below the water table. If such construction is required at a particular site, another site should be considered. When another site is not available, the structure should be located above the water table and extensive fill, to provide radiation protection and complete burial, should be placed over the structure.

Since time will not allow rock excavation, a structure must be located on the surface of the rock and extensive fill placed over it where bedrock cannot be avoided. Of course, it is desirable to seek an alternate site where rock will not be encountered.

2.1.6 Requirements for Entrance Structures. The entrance to the shelter may be one of two types: (1) a vertical shaft leading to a horizontal "tube" which in turn leads into a structure; or (2) an inclined shaft leading directly into the structure or to a horizontal "tube" projecting from the structure. For either type, added radiation protection, if required, can be provided by sand-bagging the entrance "tube" where it enters the structure. Also both types of construction will require a tube as the actual entranceway with a hatch to prevent the shock from entering the structure. The support for this hatch must be so arranged to isolate the large reaction of the closure from the entrance tube. In this report only the vertical shaft leading to a horizontal passageway which, in turn, leads into the structure is considered.

2.1.7 Ventilation Structures. The ventilation system must serve two primary purposes: (1) cooling the interior of the structure, and (2) replenishing oxygen and removal of carbon dioxide. By virtue of the opening introduced into the structure by the presence of the ventilation system an incidental problem, that of preventing the shock from entering the structure, presents itself. One solution to the entire problem would be provision of air conditioning and chemical or mechanical means of producing oxygen and removal of carbon dioxide. However, chemical methods for generating oxygen and absorbing carbon dioxide require relatively large items of equipment which implies increasing the size of the basic structure and also a possible problem in transporting the equipment. A more economical and simpler solution appears to be possible by providing one intake and one exhaust structure for each 51-man shelter. Alternatively where several structures are located adjacent to one another, a single intake and single exhaust system, of larger size, can be provided to accommodate all of the structures.

2.2 ENGINEERING FACTORS

2.2.1 Material, Structural Type, and Site. In the general case of designing a protective structure, the problem resolves itself essentially to one of economics. This requires the study of several possible alternative types of construction including the use of several different structural materials and also the locations of the structure. In the case of the problem primarily considered herein the economics cannot be neglected, but because of the limitations of time and available equipment, certain types of construction must be eliminated from consideration. The available equipment necessitates keeping the weight of each individual element of the

structure as light as possible. The available time for construction and immediate occupancy of the structure precludes the use of cast-in-place concrete and of using timber which might be available immediately at the site. These limitations of time and equipment therefore required consideration of a structure consisting entirely of metal elements, entirely of timber elements, entirely of precast-concrete elements, or a combination of these.

Study of these possible means of construction led to the conclusion that the entire structure could be provided best to an engineer platoon by including all of the parts as a kit which could be transported by flatbed trucks or even air-dropped. For the single personnel shelter to be located in a forward area, it appears the most suitable structure will consist of steel arch ribs which support timber blocks as simply-supported beams spanning between the ribs. Each complete rib to be fabricated in two parts which when field-connected would form a 180 degree circular arch with an internal radius of 8 ft. These ribs are supported on timber sills reinforced along each side by steel channels. The steel channels form a field splice of the timber sills thus permitting the sills to be supplied in 12 ft. lengths. End bulkheads for this structure consist of a wide-flange-beam posts supporting timber blocks between their flanges. The wide-flange posts in turn are supported by a horizontal truss spanning between the sills. Detailed plans for this structure are included as Appendix J.

In addition to the advantages already noted for the type of construction described above, this structure contains the simplest possible connections of all of the structural types considered. This structure also

gives a minimum number of different structural members of the possible structural types considered. It lends itself very well to modular construction, with any multiple of 12 ft. being a possible length of the structure.

The only disadvantage of this structure which has been encountered is the difficulty of backfilling. Since this arch depends upon the backfill to mobilize its design strength, the backfilling of such a structure is more critical than the backfilling of a structure with greater inherent flexural strength. However, except for the fact that heavy mechanical equipment cannot be used for compaction, this backfilling does not present any major difficulties, especially since there will be a relatively large force of man power to accomplish it and since availability of heavy equipment may not be counted upon.

2.2.2 <u>Design Overpressure and Loading</u>. It is likely that personnel would be given less importance by an enemy than heavy weaponry or communication centers. As a result personnel shelters generally would not be designated ground zero for the detonation of large yield nuclear weapons. By dispersing personnel shelters a reasonable distance from the probable prime targets, these shelters can be designed to resist overpressure levels of tolerable proportions.

From a defensive standpoint the distance by which a personnel shelter should be removed from designated ground zero, and the corresponding design overpressure level, are inherently related to the size of weapon against which protection must be provided. Also the probability of a certain overpressure level being attained depends upon the circular probable error (CEP) compatible with the method used by the enemy to deliver the weapon. Knowing the size of the weapon and the associated CEP provides a means of

establishing the probable distances from ground zero at which certain overpressure levels will be exceeded as well as the probability of these levels being exceeded. The size of weapon and method of delivery cannot be accurately forecast, but for the nuclear weapons which would probably be employed, it does not seem practicable to disperse beyond a range which would correspond to an overpressure of 100 psi. Therefore, this should be the minimum pressure considered in the design of personnel shelters, and consideration of higher overpressures well may be advisable. The proper overpressure to be considered for a specific design (or on the other hand the distance by which a structure must be removed from other targets) must be left to the designer who, it is expected, will have available the types of units in a particular military deployment, the weapon capability of the enemy, and means of establishing CEP's for this particular capability.

Once the overpressure level for which the design must be made is established, the loading acting on each structural element may be estimated from the data given in Chapter 4.

2.2.3 Mode of Failure and Factor of Safety. For reasons of economy it is essential to use ultimate strength concepts to accomplish the design. These concepts require the visualization of the possible modes of failure of the structure, and provision of sufficient strength to eliminate these modes of failure. The means by which this is accomplished are given in Chapter 4.

The factor of safety is chosen by the designer when he chooses the overpressure to be protected against. That is, by following the methods briefly outlined in the preceding section the designer determines the probability that a particular overpressure is going to be exceeded. For large

overpressures it generally is not practical to choose a zero damage probability as an objective since this would result in almost fantastic design requirements. Yet a small concession in this damage probability generally decreases the design requirements considerably. As a result if a damage probability of 20 percent, for example, is assumed and the structure proportioned to resist the corresponding overpressure, there would be an 80 percent probability of survival. Obviously absolute safety would not be provided in this case, but the provision of absolute safety of all who might be involved in a nuclear attack cannot be economically justified and a calculated risk must be taken as in any battle situation.

2.2.4 Ductility Ratio. In addition to using ultimate strength concepts it is desirable to allow a maximum deflection x_m of an element, in excess of the yield deflection x_y. The ratio of x_m to x_y is referred to as the ductility ratio μ. Ductility ratios greater than 1.0 allow part of the applied force to be absorbed by plastic deformation of the element. The amount of plastic deformation which might be allowed depends upon the ductility available in the material or the structural element. The maximum ductility ratios for the various materials considered are recommended in Chapter 4.

2.2.5 End Restraint for Particular Members. In general for protective construction it is desirable to have members continuous over as many supports as possible because of the reduction in moments effected thereby. However, such practice for the type of structure emphasized in this report is not desirable for two reasons: (1) the length and weight of continuous members would be contrary to the general criteria which must be met; and (2) use of continuous members would place serious restrictions upon what modular construction might be possible.

2.2.6 Foundation Type. Again a concession to normal design practice was necessary to fulfill the objectives of this program. Normally foundations would be cast-in-place concrete using a trench for forming to assure continuous support. Since cast-in-place concrete could not be used in the time allowed, timber sills reinforced with steel channels appear to be the best alternative.

2.2.7 Spacing Between Structures. There are two major factors to be considered in choosing the proper spacing between structures: (1) the structures must be far enough apart that the presence of one does not affect adversely the loading on the other; and (2) by dispersing identical units sufficiently, the overpressure for which the units must be designed can be reduced and the probability of at least one survival can be as high as that of a single unit of much greater strength. A more complete discussion of the minimum spacing to insure proper structural action, under item (1) above, is presented in Appendix G.

2.2.8 Interaction of Main Structure and Appurtenant Structures. It appears that the entrance and ventilation structures should be installed in the end bulkheads or in the passageway where the necessary opening can be more readily framed to provide the necessary strength than can be done in the arch of the main structure. There is an additional advantage in that it is easier to fabricate these appurtenant structures to attach to a plane surface than to a curved surface. Furthermore, with the entrance at the end, the arrangement of equipment within the structure is more efficient.

3. FACTORS INFLUENCING ARCHITECTURAL DETAILS

3.1 ADVANTAGES OF BURIED CONSTRUCTION

In this report the use of buried construction has been implicitly assumed. This assumption is the result of many considerations which are enumerated below.

3.1.1 Characteristics of Loading. By completely burying a protective structure the antisymmetrical loads are reduced while the deformation produced by them is nearly eliminated. Therefore, the cost of excavation, of possibly waterproofing and of artificial ventilation, required in a buried structure, may be offset by the saving in structural materials. The overpressure at which these costs balance depends upon the particular site conditions. However, an overpressure of 30 psi or more would appear to require buried construction from the economic standpoint in any case, unless the size of weapon for which protection is provided is much less than the minimum operational sizes currently considered. Since in this study a design for an overpressure of at least 100 psi was required, it was obvious that only buried structures should be considered.

3.1.2 Radiation Protection. Requirements for radiation protection make buried construction very attractive. Radiation normally is divided into two categories, prompt and residual. Under the design criteria established by WES the occupants of the structure were to receive a cumulative dosage from both sources of 25 Roentgens (R) during the specified [illegible]k shelter occupancy. Obviously the relative amounts of radiation [illegible]ble to the two categories as well as the total amount of radiation depends upon many parameters. Because the study reported herein provides a standard design, it is impossible to specify all of these

parameters. Yet, the amount of cover required for structural requirements apparently is sufficient to provide sufficient radiation protection. Furthermore, it generally is more economical to balance the cut and fill required for the structure to avoid the necessity of disposing of any excess cut or of borrowing material for fill. Considering this balancing of the excavation indicates that the minimum cover required to accomplish full burial can be provided economically.

From considerations of structural integrity, 4 ft. of cover was placed over the crown of the arch. In Appendix D an analysis is presented of the radiation protection afforded by the structural configuration adopted. Because the standard nature of the basic design did not allow definition of a specific input in terms of weapon yield and point of detonation, several conditions are assumed in the appendix. For the near surface burst, it is concluded that a protection factor of: (1) more than 20,000 is provided for residual radiation; (2) more than 3000 is provided for prompt gamma for a mean spectral energy of 4 mev.; and (3) more than 20,000 is provided for prompt neutrons. For a burst immediately above the structure at a range sufficient to produce a 100 psi diffraction pressure on the surface, these protection factors are greatly reduced. However, it must be noted that there is a low probability of a burst occurring within the region defined by the solid angle corresponding to these drastically reduced protection factors, so that the over-all survival probability of the basic shelter is not compromised by reduction in protection factors attending an overhead burst.

3.1.3 Terrain Factors. A buried structure, especially one below the original ground surface, may be concealed provided that extreme

care is taken to blend the backfill with the surrounding ground and to camouflage the entrances and ventilation structures. A structure founded on the original ground with an extensive earth mound over it may be similarly concealed.

Terrain may also influence the construction methods employed. Buried structures probably will be constructed by one of two methods, tunneling or cut-and-cover. Tunneling is feasible only where there is sufficient topographic relief to allow construction of a nearly horizontal tunnel. If the terrain is relatively flat, the structure must be cut-and-cover construction. This results from the fact that so long as a structure is located outside the zone of rupture of the ground in the vicinity of ground zero, there is no necessity for placing the structure deeper than is required to satisfy complete burial and radiation requirements. Some advantage may be gained as a result of attenuation of the shock with depth below the ground surface; however, the depths necessary to develop significant attenuation are such that it is not economical to strive for this advantage by cut-and-cover methods. On the other hand, where tunneling is possible this advantage might be economically gained. In this regard however, the designer should be warned that the entrances of a structure are its most vulnerable part; being deeply buried may increase the vulnerability of the entrances by virtue of the increased length of the entranceways.

3.1.4 Location of Water Table and Bedrock. The locations of the water table and of bedrock are very important considerations in choosing the site of the structure and the type of construction. Where the water table or bedrock is near the surface such that a structure located completely

below the ground surface may necessitate excavation below the water table or into the rock, another site should be considered. Another site should be considered because of the many advantages of locating a structure entirely below the original ground. Where construction on a site is absolutely required and the bedrock or the water table is near the surface, there is no choice other than placing the structure above the bedrock or water table and covering it with extensive fill from borrow pits since the basic structure is not designed to be placed below the water table and since an engineer platoon could not, in the time available, undertake rock excavation. When the structure must be located above bedrock at least one ft. of fill must be placed between the rock and the sills.

3.1.5 Entrance Requirements. Because of the short warning of imminent attack, the entrances must be simple enough to allow fifty men to enter the structure and "button it up" in twenty minutes or preferably less. Thus, the entrances must be simple in operation, yet sufficient to resist the expected overpressure and radiation. The cover for the entrance must have sufficient strength to resist the expected forces, yet it must be light enough to be placed in position manually and fastened within something less than five minutes.

Additional radiation protection at the entrance if required, can be accomplished most easily by filling the end of the passageway at the point where it enters the structure with sand bags which were filled during the construction operation. However, it was apparent in the computations summarized in Appendix D that the radiation entering through the entranceway is negligible. Consequently, it is believed that the additional protection in the entranceway will not be required.

The simplest type of entrance structure is a round vertical pipe with a hinged door or closure. The support for the closure or door must be isolated from the entrance tube to avoid collapsing the tube. This isolation may be accomplished by laying a sill around the entrance tube and supporting the closure on this sill. This sill must be anchored by use of dead men so that it will not lift out during the negative phase of the air blast or as a result of elastic rebound of the closure. The access door also may be supported on columns resting on the entrance structure below.

3.1.6 Ventilation Requirements. Ventilation of a protective structure presents many problems among which are: (1) prevention of blast from entering the shelter; (2) filtration of radioactive particles from incoming air; and (3) insuring the proper flow of air to maintain habitability. Preliminary investigation indicated a Swedish rock grille of the type tested in Operation Upshot-Knothole (3.3) would provide a ventilator compatible with the original design requirements. Subsequent investigation, based primarily on unpublished test results from USNCEL, indicates that the Swedish rock grille can mitigate shock effects from short duration pulses (of magnitude associated with conventional HE weapons) to tolerable levels, but these devices are not adequate for pulses with pulse durations associated with nuclear weapons and "low overpressures" (less than a few hundred psi).

Therefore, other devices were considered to provide the necessary closure for the ventilation system. The investigation was hampered by the fact that the revised criteria for the standard structure did not allow definition of the ventilation requirements. That is, although the basic structure was to house 51 persons, it was impossible to specify, for the

general case, how much equipment would be housed in the structure. For that matter it appears that the standard equipment has not yet been specified for the shelter to be used only in a forward area by an engineer platoon. Additionally it is conceivable that for some installations, primarily civilian shelters in regions of high land use and involving combustible construction a fire storm might develop after an attack; in such an eventuality air-conditioning probably would be required. Consequently, it was apparent that in many cases, primarily a platoon shelter in a forward area, a simple ventilation system consisting only of intake and exhaust ducts, simple blast closures, and a fan to overcome head losses would be required; on the other hand completely encapsulated systems with bottled oxygen, CO_2 scrubbers, air conditioning, etc. might be required for installations where development of a fire storm was likely.

Much deliberation of the problems associated with ventilating a standard structure led to the conclusion that ventilation could not be standardized; each shelter and the equipment for it must be analyzed to determine the ventilation requirements and the type of hazard which is likely to develop. Knowing these allows specification of the components required in the ventilation system. To allow for the many situations which might have to be accommodated in the ventilation system, the standard shelter and passageway can easily be expanded to have the necessary ventilating components. Generally it was considered preferable to place these components in the passageway or its extension, the utility structure, because (1) the noise and potential fire hazard associated with the fans and gasoline driven power unit could be isolated from the main structure; (2) contamination associated with the use of a CBR unit would be isolated

from the main structure; and (3) the relatively small area associated with each component and the flow pattern through the equipment indicated greater economy of construction by placing these units in a long-narrow structure than in the main structure. Accordingly the passageway and utility structures were designed to accommodate directly the standard Chemical Corps M-9 CBR unit which appears to have more than adequate capacity for the platoon shelter in a forward area. With some modification a standard M-10 CBR unit can be accommodated in the passageway. Where larger flows are required several of these units can be placed end to end along the passageway or a larger standard unit might be modified to fit into the passageway.

Similarly the intake and exhaust ducts for the standard structure were specified to accommodate a large range of flow rates to account for the conditions which might prevail for any condition. These ducts were designed to provide a maximum air velocity of 40 fps, which is consistent with the specifications for the CBR filters and also for standard blast valves which are available. A particular type of blast valve is not specified in the standard design because of: (1) unknown requirements for flow rate in the general case; (2) unknown requirements for means of actuating the values; and (3) apparent lack of a commercially available blast valve of the relatively small capacity required for the platoon shelter in a forward area.

To determine the quantity of air which must be provided to maintain the 51 men in the standard shelter several factors must be considered. Air is required to replenish the oxygen (O_2) in the shelter, to remove the carbon dioxide (CO_2), to remove the water vapor (H_2O), and possibly to remove the heat generated by the occupants. According to Ref. (3.4) each

occupant requires or generates per hour the amount of gases or heat shown in Table 3.1. In Table 3.1 the data from Ref. (3.4) are compared with minimum values based upon the data from Ref. (3.5). In Ref. (3.5) the O_2 requirement is given as 1 liter for every 5 calories of food intake. Also in this last reference it is stated that the normal male performing average amounts of labor requires 3000 calories per day. Using these latter data yields an O_2 requirement which averages 0.875 ft.^{3}/hr. which compares very favorably with the demand given in Ref. (3.4). Thus, it is believed that Ref. (3.4) assumes normal activity of the shelter occupants. However, once the personnel shelter is occupied, the physical activity of the occupants, by necessity, will be greatly curtailed. Also there will not be sufficient food nor a demand for a diet of 3000 calories/day. Therefore, it seems more reasonable to base the volume of air required upon an average food intake of 2000 calories/day.

On this basis the volume of air required, and the interval at which the air must be changed within the structure was computed; the results of these computations are shown in Table 3.2. The assumptions used in computing the values in Table 3.2 are summarized as a footnote to the table. These assumptions appear to be self-explanatory except for the one concerning no heat conducted into the surrounding soil. Normally soil a few feet below the original ground surface in a temperate climate would be at a temperature of approximately 55°F. For other climatic conditions the subsurface temperature equals the mean annual air temperature. However, in the standard design being considered here, the shelter is to be occupied immediately after completion of construction. Also, it being a standard design, it is conceivable that some shelters will

be constructed in sub-tropical or tropical climates, and the natural soil and the soil used to backfill the structure, by virtue of its being exposed during the construction phase, may attain a temperature considerably in excess of 55°F. Therefore, for the standard design considered here, it did not seem proper to allow conduction of heat to the surrounding soil which may not be capable of absorbing significant amounts of heat. Yet this is not particularly conservative since it has been assumed also that all latent heat is carried off with the water vapor.

There are two significant facts to be observed in Table 3.2. First is the very large volume of air required to maintain a tolerable temperature. This will be discussed at greater length below. Second is the fact that neglecting the heat gain, the structure can be sealed completely for a period of 4.9 hours without causing any severe effects among the occupants. This ability to close off the structure from the outside is extremely desirable in the event that a fire-storm situation develops. Under such a situation the entire area in and around the storm is rendered completely devoid of oxygen for varying periods of time, but probably for a time less than 4.9 hours. Consequently, should such a storm develop in the vicinity of a shelter, it is imperative that the shelter be absolutely sealed. If means are added to remove chemically the CO_2, the shelter could operate up to 9.4 hours absolutely sealed.

Because the major volume of air supplied to the shelter results from the necessity of cooling the interior and because during a fire-storm this volume of air cannot be supplied, it is necessary to consider other means of cooling the interior of the shelter. The 150 BTU/man-hr. of enthalpy generated, allowing a 20 percent increase for equipment, corresponds

to 9200 BTU/hr. of air conditioning needed to prevent the average internal temperature from exceeding approximately 90°F. Since 12,000 BTU/hr. of air conditioning requires theoretically 4.7 horsepower, at least 3.6 horsepower must be provided by a combination of mechanical energy and a waste heat sump.

As an alternate 5.4 tons of ice could be stored in the structure which theoretically would supply cooling over the one-week's occupancy equivalent to the air conditioner. The ice, however, appears to be a relatively inefficient means of cooling the entire structure. Also storage and drainage would have to be provided to use ice for cooling. Storage of ice would require 170 $ft.^3$ which is approximate 3 percent of the gross volume of the proposed main structure. As a result, it is believed that the air conditioner is the more reasonable means of supplying the necessary cooling although an ice reservoir may be an efficient heat sump for use in combination with the air conditioner.

3.1.7 Power Requirements. The requirements for power for running the air conditioning system already have been mentioned. In addition power must be provided for overcoming the head loss in the ventilation system and for operating lighting and communications equipment within the shelter. Because, as already discussed, it is impossible to specify a single ventilation system, it is impossible to specify the power required. Computation of power requirements is relatively simple after the ventilation system is designed.

3.2 BASIC STRUCTURAL TYPES

The different structural types considered during this investigation were: (1) arches, including ribbed and structural plate, (2) barrel or pipe "arches," and (3) flat roof; including slabs and beams and girder. In general any of these three types may be fabricated from several possible materials. However, because of limitations imposed by the availability of time, equipment, and experienced fabricators, some materials have definite advantages over other materials. These limitations are discussed first in the following section.

Because the structures considered in this report must house personnel, there is no necessity for large clear spans within the structure. This observation allows considerable latitude in the choice of structural type since interior columns can be placed at practically any point in the structure although a psychological problem may result from cluttering the interior of the structure. The clear span required affects the type of construction chosen and some discussion of this subject is included following a discussion of the specific space requirements involved.

Finally, foundation and backfill requirements for certain structural types may influence the kind of construction required. Therefore, this subject is discussed also among the advantages of the possible types of construction.

3.2.1 <u>Practical Limitations</u>. An engineer platoon in the field primarily has available hand tools and a fairly sizable labor force. However, they are limited in the size and capacity of mechanical equipment. At best they will have a bulldozer, a truck with a winch, and for limited periods a light power shovel with a crane boom may be available from

battalion headquarters. This power shovel might be available to each company for a maximum of two days per week. It is imperative for the engineer platoon to complete and occupy its protective structure within one week or less. As a result the power shovel may be used along with the bulldozer to complete the excavation for the structure, but it probably will not be available for actual construction. Therefore, actual construction must be accomplished mainly by manual effort, and the structural type chosen must be compatible with this means of erection.

Since heavy equipment cannot be counted upon even for making the excavation, it is desirable to find an alternate method for quickly making the cut in which the shelter is to be located. It appears that making this excavation can be expedited by placing three 100 lb. TNT charges along the longitudinal centerline of the cut for the shelter. These charges should be placed at a depth to the center of the charge of 4.7 ft. with one charge set immediately over the center of the expected location of the structure. The other two charges should be set 14 ft. along the centerline from the charge at the center. It appears that these three charges will complete about 17 percent of the total excavation and that they will effectively loosen all soil which must be removed.

If it is assumed excavation will require two days to complete, approximately two days will be available for complete fabrication of the structure since backfilling will probably require three days for completion. Thus a structural type must be chosen to allow complete fabrication within a period of two days or less. For this reason the structure must be prefabricated and the structural details must be simple enough to allow rapid field connection.

To reduce the cost and to eliminate the necessity of field-sorting of parts, the greatest possible duplication of structural details must be utilized.

3.2.2 Space Requirements. It is the purpose of this portion of the report to present the general layout or plan and the various types of structures that were considered.

A typical floor plan for a minimum shelter for 51 men is shown in Fig. 3.1. The width depends upon whether the structure has a flat roof and vertical sides or is some type of an arched structure. Figure 3.1 indicates the widths for the various types of structures. The actual floor plan will be influenced by the location of the entrances or accessways.

A typical cross section, illustrating the proposed bunk arrangements, are shown in Figs. 3.2 and 3.3. Various types of bunks have been considered. In each case one of the prime considerations was a type of bunk that could be readily moved aside or folded when not in use. A hanging bunk appears to be more suitable than a folding bunk. The cross sections shown were all selected on the assumption that suspended bunks would be used.

As previously stated, Fig. 3.1 depicts what might be called a minimum shelter. In addition to sleeping quarters, it has been proportioned to provide for the items listed below. Work space or other additional space could be provided by lengthening the over all structures as the contemplated structures [illegible] of a modular type, and each module is designed to be self-sufficient insofar as structural integrity is concerned.

(1) Drinking water. Space has been provided for water based on a water consumption of two quarts per day per person. Four 55-gal. drums per week thus are required for 51 occupants.

(2) Sanitary facilities. Space is provided for a chemical toilet system with two stools and a 200-gal. elliptical tank. Additional capacity could be obtained by hand-pumping to water barrels after they are emptied.

(3) Food. Approximately 16 sq. ft. have been provided for food boxes and chests.

(4) Electrical power and communications. A total space requirement of approximately 30 sq. ft. has been arbitrarily assumed for this item.

(5) Recreational area. It has been assumed that this area would be provided by raising the suspended bunks.

Principal consideration was given to three types of cross sections. These may be generally described as follows:

Type I -- Arched type structures

Type II -- Corrugated metal pipe arch

Type III -- Flat roof and vertical sides

Figure 3.3 shows the three general types of structures with bunk locations for each type. Table 3.3 indicates the area provided by each of these structural types. Various adaptations of Type I received considerable attention. The most promising of these were further subdivided as follows:

Type I-A -- Steel ribs consisting of inverted structural tees

Type I-B -- Corrugated metal plate arch

Figure 3.4 shows Type I-A in some detail. Timbers and corrugated metal plates are shown as possible wall sheeting. The timbers appear to be more suitable because they inherently add longitudinal stiffness to the structure to support the end bulkheads. At the higher overpressures the edges would require beveling in order to augment the strength of the arch. Type I-B is similar to the commercially available Multi-Plate arch manufactured by Armco.

Type II is a corrugated metal type similar to Armco's Multi-Plate arch. As shown in Fig. 3.3, a clear span of approximately 14 ft. 3 in. was contemplated. As a result of the analysis presented in Appendix F both Type I-B and Type II must be discarded since thicknesses of corrugated metal currently available do not possess sufficient strength to withstand an overpressure of 100 psi. Even if heavier corrugated sections were obtained they probably would not be adequate in resisting the thrust induced by the end bulkheads.

Type III lends itself to several adaptations. One of the most promising possibilities for this type is shown in Fig. 3.5. The ceiling beams and the columns are steel I-beams. Both timber and corrugated metal are shown as possible sheeting materials. A girder and columns may be used along the longitudinal centerline to reduce the size of the roof beams.

3.2.3 Arch Construction. The major disadvantage of any arch construction is the inherent instability of the arch until it is backfilled. This condition results because the arch depends upon the backfill to mobilize its strength. For this reason backfilling is a critical operation.

Because of the unstable condition of the arch, heavy equipment cannot be operated in the near proximity of the arch. Thus most of the backfilling operation must be accomplished by hand. However, because of the relatively large amount of manpower available, meeting the manual backfill requirements should not be difficult. In order to secure the maximum benefit of arch action, backfill should be hand tamped in approximately 6 in. lifts with the soil at optimum moisture content. It has been estimated that the available 51 men should be able to place this backfill within the three days allotted.

The major advantage of the arch is that when symmetrically loaded it has a compressive stress only. This characteristic of the arch permits use of a clear span and simplified connections since only compression connections are involved.

The arch might be either the barrel or ribbed type. Since cast-in-place concrete cannot be utilized because of time and equipment limitations, the arches must be fabricated of timber or precast concrete and/or metal. All-timber construction is not feasible because of the large proportions which are required and because of the curved members required. Thus, the arch cross section is limited to a consideration of a barrel arch of corrugated metal or a ribbed arch with metal ribs supporting corrugated metal, timber, or precast concrete elements. Because of the relatively small stability inherent in a barrel arch of corrugated metal, especially in the construction phase, the ribbed arch is much more easily fabricated and backfilled. Also as indicated in Appendix F corrugated metal arches without stiffeners are limited to conditions requiring protection from relatively low overpressure. This reason alone is sufficient to eliminate the pipe or barrel arch from consideration. However, it also has other disadvantages; (1) the sections of a barrel arch become relatively large; and, consequently, they are unwieldy to handle without mechanical equipment, (2) the relatively large number of bolts required in a corrugated metal barrel arch would require too much time to place, and (3) there is comparatively little strength transverse to the corrugation.

Because of the desirability of limiting the weight of individual elements for the single personnel shelter to be fabricated in a forward area, the timber blocking is the preferable material available for spanning

between the inverted structural tee ribs of the arch. Corrugated metal must be ruled out in any case for the configuration of shelter recommended herein for it will not provide sufficient longitudinal strength to resist the reactions from the end bulkheads. Since as illustrated in Appendix H timber blocking will resist overpressures on the surface as high as 300 psi, it would appear that precast concrete blocks to span between the ribs of the arch need be considered only where the procurement of large amounts of timber is difficult. Thus, it is recommended that timber blocking be used to span between the arch ribs.

3.2.4 Beam and Girder Construction. This type of structure has two major advantages; (1) backfilling can be easily accomplished and (2) the structure has the longitudinal strength to transmit end bulkhead loads to the opposite end. Since the members of such a structure are designed primarily to resist the shock loads in flexure, backfilling is not critical and heavy equipment generally may be used for compaction.

Also, as already mentioned, there is no structural requirement for large clear spans for the type of construction considered herein. Therefore, by using interior columns and longitudinal girders the span of the roof may be 1/2 or less of the total span required. Considerable savings in the material thus may be obtained by reducing the clear span of the roof.

Preliminary designs indicated, however, that even when two interior columns were assumed the material required exceeded that needed by various types of arches. The flat roof beam and girder construction is not amenable to easy field fabrication as the connections are awkward and massive. Furthermore, interior columns necessitate some type of footing within the

shelter. Comparative designs of ribbed arch and beam and girder shelters indicated that considerably more material was required for the steel roof beams than for the arch ribs. The material required for the interior columns and girders were a completely additional requirement. Thus the arch was found to require less material and to be much simpler to erect than a comparable rectangular structure.

Only a composite structure consisting of steel columns and beams supporting timber, corrugated metal or precast concrete elements was considered in the qualitative comparison of weight immediately above. Of course it would be possible to construct a rectangular structure, of the size being considered, entirely of timber; i.e., timber columns and timber beams and stringers supporting timber sheathing. However, for the overpressures which must be considered, the dimensions of the individual timbers would be of such magnitude that obtaining the lumber might be impossible. For example, if stringers of 7 ft. span were spaced 2 ft. on centers, the required section modulus for protection against an overpressure of 100 psi would be approximately 360 in.^{3} if stress grade lumber were used, and the required section modulus would increase linearly with the overpressure. A solid timber deck of 7 ft. span to resist 100 psi would be approximately 6 in. thick and the longitudinal beam along the center of the structure supporting this solid deck, if columns 7 ft. on centers are assumed, would be approximately 6 x 19 in. in actual cross section. Such a timber structure could be fabricated but probably not within two days. Also it is obvious that steel columns, beams, and girders would be more economical.

3.2.5 End Bulkheads. Several different configurations of end bulkheads were considered. Among those considered were:

(1) A quadrant of a hemisphere using inverted structural tees supporting timber blocks to form the hemisphere.

(2) A heavy wide flange cross-beam spanning between the ends of the sills. This cross-beam would carry the lower reaction of the vertical bulkhead posts. The upper reaction of the wide flange posts would be carried from one end of the shelter to the other by the blocks between the arch ribs.

(3) A horizontal or nearly horizontal steel arch fastened to the ends of the sills. The lower end of the vertical bulkhead posts would be tied to the arch by use of cables.

(4) A system of dead men to support the lower end of the vertical bulkhead posts.

(5) A prefabricated truss spanning between the ends of the sills. The truss serves the same function as the wide flange cross-beam in (2) above.

Type (1) was discarded because of the heavy weight of a bearing block required at the top to support the radial arch ribs. Similarly (2) was discarded because of the excessive weight of the cross-beam, and (4) was considered to be inappropriate for field construction by inexperienced troops and to require excessive mass in order to provide the dead man resistance.

A complete design was prepared for the horizontal arch. However it was found that to facilitate the erection of the vertical posts it was necessary to place a light weight cross-beam in order to support the vertical posts during the erection process. It also was determined that the details necessary to accommodate the standard cables were quite expensive.

Of all the types considered the light weight truss proved to be the most suitable. The major advantage of the truss is that it is a complete unit and may be placed as such. It furthermore serves as a tie between the ends of the sills.

3.2.6 Foundations. Because the use of cast-in-place concrete was precluded by the time requirements, the design of the foundations presented some difficult problems. However, consideration of ease of field fabrication and keeping weight a minimum suggested the use of timber for the sills supporting the shelter. For ease of handling it was decided to limit the length of the individual sections of the sills to 12 ft. To reduce differential settlement between adjacent ends of the sill sections, steel channels are bolted on either side of each sill. Thus the timber forms a splice for the channels and the channels form a splice for the timber.

3.2.7 Entranceways and Utilities. As stated previously only a vertical personnel entrance structure was given primary consideration in this report. A brief discussion of the problems associated with other types of entrance structures is given in Appendix E.

The principal advantages of the vertical type entrance structure in comparison with the walk-in type structure are economies of materials and time required for installation and an increased facility in the installation of the lighter weight vertical entrance structures. While these advantages are real it must be noted also that the vertical entrance structure is not well suited to the passage of equipment into the shelter nor for the rapid ingress or egress of personnel.

There are two possibilities for joining a vertical entrance to a shelter; (1) it may be connected directly to the shelter itself, or (2) it may be connected to an intermediate entranceway that is connected to the shelter. Either of these alternatives may be placed at the end of the structure or at the side of the structure. In the shelter under consideration, the end framing entrance was the more desirable for the following reasons:

(1) It is much simpler to frame into the vertical plane surface of the end bulkhead, than into the curved, ribbed arch of the shelter itself.

(2) There is less loss of interior space by framing into the shorter dimension of the structure than into the longer dimension.

After study it was decided not to frame the entranceway directly into the end of the shelter but to use an intermediate passageway instead. The intermediate passageway concept offers these advantages: (1) simpler connection achieved by separating the entranceway from the shelter itself; (2) prevents the loads that are applied to the access door from being transmitted to the shelter, by having a rather flexible passageway; (3) reduces the radiation entering the shelter through the passageway; and (4) passageway may be used for other purposes, such as location for decontamination station or for mechanical equipment.

The entranceway and passageway for the standard structure evolved over the entire period of the design of the shelter. At first a simple vertical tube including both rungs and a firepole leading to a short length of horizontal conduit, which in turn passed through the end bulkhead of the main structure, was considered adequate. The vertical

tube and horizontal conduit were proposed to be fabricated from standard corrugated steel plate with corrugations of depth 2 in. plus the plate thickness. Study of this solution led to the finding that corrugated steel plate with the heavy corrugations required for flexural strength could not be formed to a diameter less than 60 in. without using special dies for forming. Further investigation led to the conclusion that the required special dies did not exist, and to fabricate them, while maintaining a reasonable cost for the end item, would require a guaranteed sale of several thousand tons of steel in the smaller diameter. Also it was concluded that entranceways fabricated of standard corrugated steel with a diameter of 60 in. did not possess sufficient strength for the required design overpressure. Further, standard presses cannot form corrugated plate from steel with a thickness greater than 1-gage.

It was found that a complicated transition piece was required to connect the circular entranceway to an elliptical passageway (The elliptical shape was required to provide some reasonable head room and yet allow the passageway to fit into the end bulkhead of the main structure.) both of which were formed from corrugated plate. Furthermore, a manufacturer of corrugated metal structures was consulted, and he was concerned about the stability of the elliptical section required for the passageway.

These considerations suggested the use of a rectangular passageway formed from welded frames of standard steel sections which supported timber lagging for the walls and steel covers for the roof. Originally timber was specified for the roof of the passageway, but the details required for fastening the timber to the steel frames suggested that simpler construction could be attained by using individual steel covers

over each panel. This form for the passageway obviated the necessity of a transition section since the entranceway could sit immediately on top of the passageway if one of the steel covers were removed. In addition this configuration circumvented the question of stability in the elliptical corrugated steel conduit.

Further consideration of the form selected for the passageway indicated that an extension of the passageway would make an ideal structure for housing of decontamination equipment which must be near the entranceway as well as isolated from the main structure and for arranging the ventilation and associated equipment since the ventilation system lends itself ideally to an in-line series. In a more sophisticated ventilation system, the intake structure precedes a fan which can be followed in turn by a CBR unit, an oxygen generator, possibly an air conditioning unit, and a discharge line into the main structure. Furthermore, operation of various equipment including the ventilation would require a self contained power source, probably an engine-generator set. An internal combustion engine must be removed from the structure to eliminate its large heat load, to prevent carbon monoxide from contaminating the structure, and to mitigate the problem of fire. Thus, it seemed desirable to extend the passageway to house these various items of equipment should they be included in the shelter.

Since the passageway must have a steel frame at either end while the extension of the passageway needs a steel frame only at one end, two kits were selected, one the passageway containing five steel frames and the other the utility structure containing four steel frames. Since only the passageway is absolutely required, while utility structures

are optional items, the end bulkhead for the passageway and utility structures was supplied as part of the passageway kit. Thus, if one or more utility structures are added at the end of the passageway, the end bulkhead is placed to cover the extreme end of the group of structures.

Finally it should be noted that the passageway (and utility structures) was designed to have a clear height of six feet. This resulted in very heavy members being required for the steel frames. Yet it became apparent that once the personnel were in the structure there was no need to maintain the vertical clearance in the passageway; in addition it appeared likely that all equipment in the passageway or utility structure could be arranged to fit in a space with only half of the six-foot clear height. Therefore, the steel frames in the passageway were redesigned assuming that a compression strut could be placed between the vertical members at mid-height. In the passageway these struts would be installed only during periods of alert so that the restricted head room would exist only at such times. In the utility structures housing equipment, these struts would be placed during construction and left constantly in position.

The general philosophy underlying the design of the ventilation structures has already been summarized in Section 3.1.6. Therefore, it seems necessary here only to comment on the basis for selection of the two configurations suggested in the detailed plans. As mentioned already with regard to the entranceway, it was found that standard corrugated steel plate with corrugations of approximately 2 in. amplitude cannot be formed economically to a diameter less than 60 in. Since cast-in-place concrete was not acceptable for the design requirements,

and since reinforced-concrete pipe seemed too heavy for this particular use, it was decided that conventional corrugated steel pipe (with corrugations with amplitude of 1/2 in. plus the plate thickness) would be the most economical section. Because of the potentially large flexural response induced near the surface of the ground, however, it was determined that standard corrugated pipe of the greatest available thickness could not be used for diameters greater than 12 in. At this point it seemed most economical to use a standard corrugated steel pipe for all ventilation tubes, and to place an annulus of concrete around the top of these tubes. This concrete serves two purposes: (1) stiffening the steel pipe near the surface for flexural loading and (2) providing a positive support for the blast valve which normally would be mounted over the opening. Use of this cast-in-place concrete compromises slightly the hardness of the structure until the concrete sets sufficiently, but this is relatively insignificant.

The door or closure must be provided with a manual lock to prevent it from being drawn open during the negative phase. Furthermore the entire door assembly must be so supported as to prevent the vertical pipe from being crushed longitudinally during the positive phase or from the whole assembly being dislocated during the negative phase.

The proposed hatch assembly is so designed that the positive phase forces are carried to the horizontal entrance structure or passageway by means of four pipe columns. Threaded rods through the pipe columns prevent the hatch assembly from being displaced with respect to the remainder of the entrance structure.

TABLE 3.1 AMOUNTS OF GASES REQUIRED OR LIBERATED AND AMOUNTS OF HEAT GENERATED BY HUMANS

Quantity	Amount Required or Generated Per Person (Ref. 3.4)*		Amount Required or Generated Per Person(Ref. 3.5)+	
	ft^3/hr	BTU/hr	ft^3/hr	BTU/hr
Oxygen (O_2)	0.85	---	0.58	---
Carbon Dioxide (CO_2)	0.70	---	0.48	---
Water Vapor (H_2O)	0.003	---	0.002	---
Latent Heat in Water Vapor	---	175	---	120
Enthalpy	---	225	---	150

*Although no criterion is given it appears from Ref. 3.5 that these requirements are based on a 3000 calorie/day diet.

+Based upon a 2000 calorie/day diet.--Only the oxygen demand is given in the reference; all other quantities computed by direct proportion which should be valid since gases and heat generated depend upon metabolism which in turn depends upon O_2 intake.

TABLE 3.2 VOLUME OF AIR REQUIRED TO PROVIDE MINIMUM HABITABILITY FOR 51 MAN SHELTER ALLOWING A 20 PER CENT INCREASE FOR EQUIPMENT*

Quantity	Volume of Air Required	Required Interval for Complete Air Change
	ft^3/hr	hr.
O_2	510	9.4
CO_2	980	4.9
H_2O	Nil	---
Latent Heat in H_2O	Nil	---
Enthalpy	26,400	0.2

*Assumptions:

(1) 2000 calories/day diet
(2) O_2 level may not drop below 14%
(3) CO_2 level may not exceed 3% (Loss of vitality can be expected)
(4) Mean daily dry bulb temperature for incoming air = 75°F
Mean daily wet bulb temperature for incoming air = 68°F
(5) Maximum interior dry bulb = 90°F
Maximum interior wet bulb = 90°F } This corresponds to

a Comfort Index = 0.4 (Dry Bulb + Wet Bulb) + 15 = 86 at which value body temperature will rise. However, since the H_2O removal is nil compared to the CO_2 removal required, the Comfort Index has not been included in the above tabulation. The significance of the assumed interior dry bulb reading, therefore, is in the computation of the volume of air required to remove the heat corresponding to enthalpy.

(6) In computing the air required to prevent an unbearable temperature increase within the shelter, it was assumed that no heat is lost by conduction into the surrounding soil.

TABLE 3.3 COMPARATIVE VALUES FOR MINIMUM SHELTER FOR 51 MEN

Type	Rise ft.	Width ft.	Total Area Sq. ft.	Sq. ft/man
I-A (Ribbed Arch)	8'-0"	16'-0"	688	13.5
I-B (Corrugated Arch)	8'-0"	16'-0"	688	13.5
II (Pipe Arch)	8'-11"	14'-3"	613	12
III (Flat Roof)	7'-6"	13'-0"	559	11

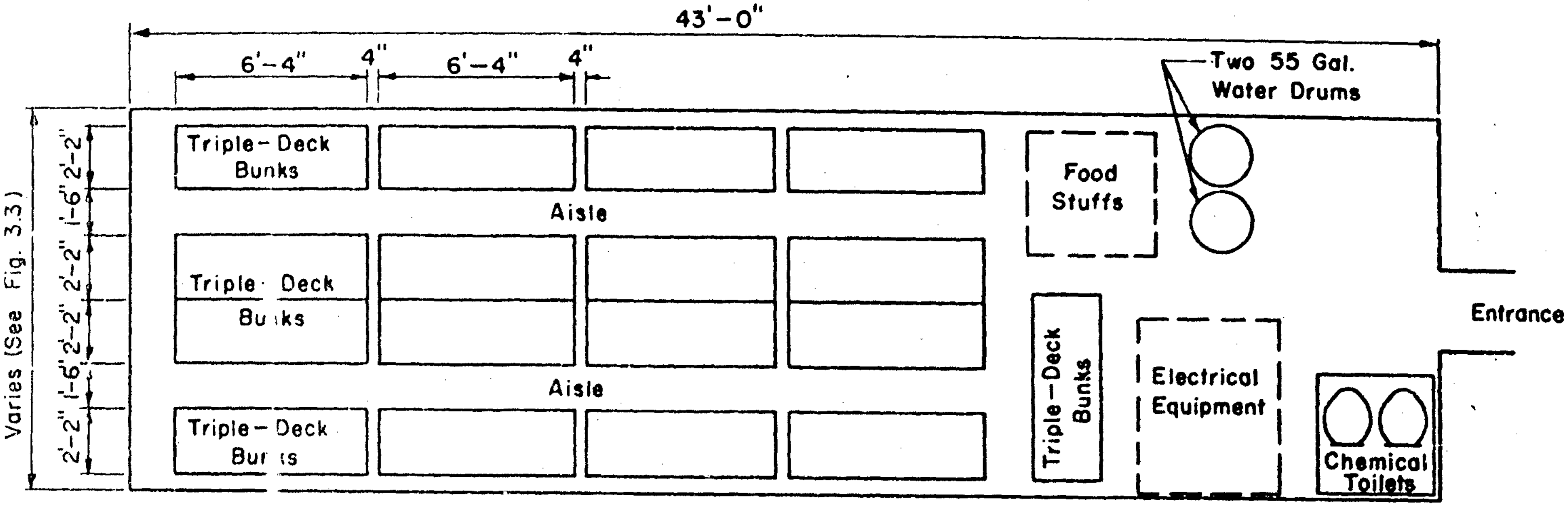


FIG. 3.1 PROPOSED FLOOR PLAN

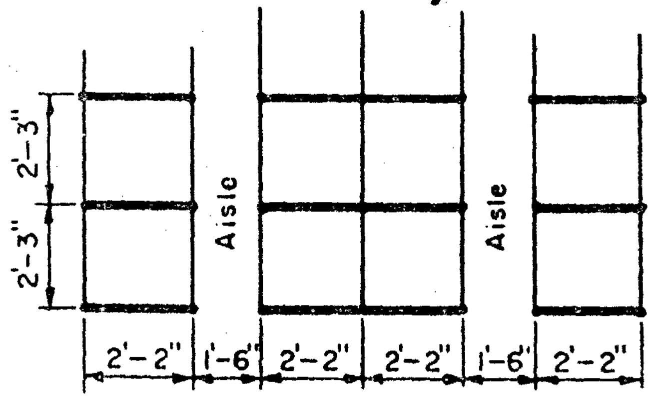


FIG. 3.2 VERTICAL SECTION THROUGH BUNKS

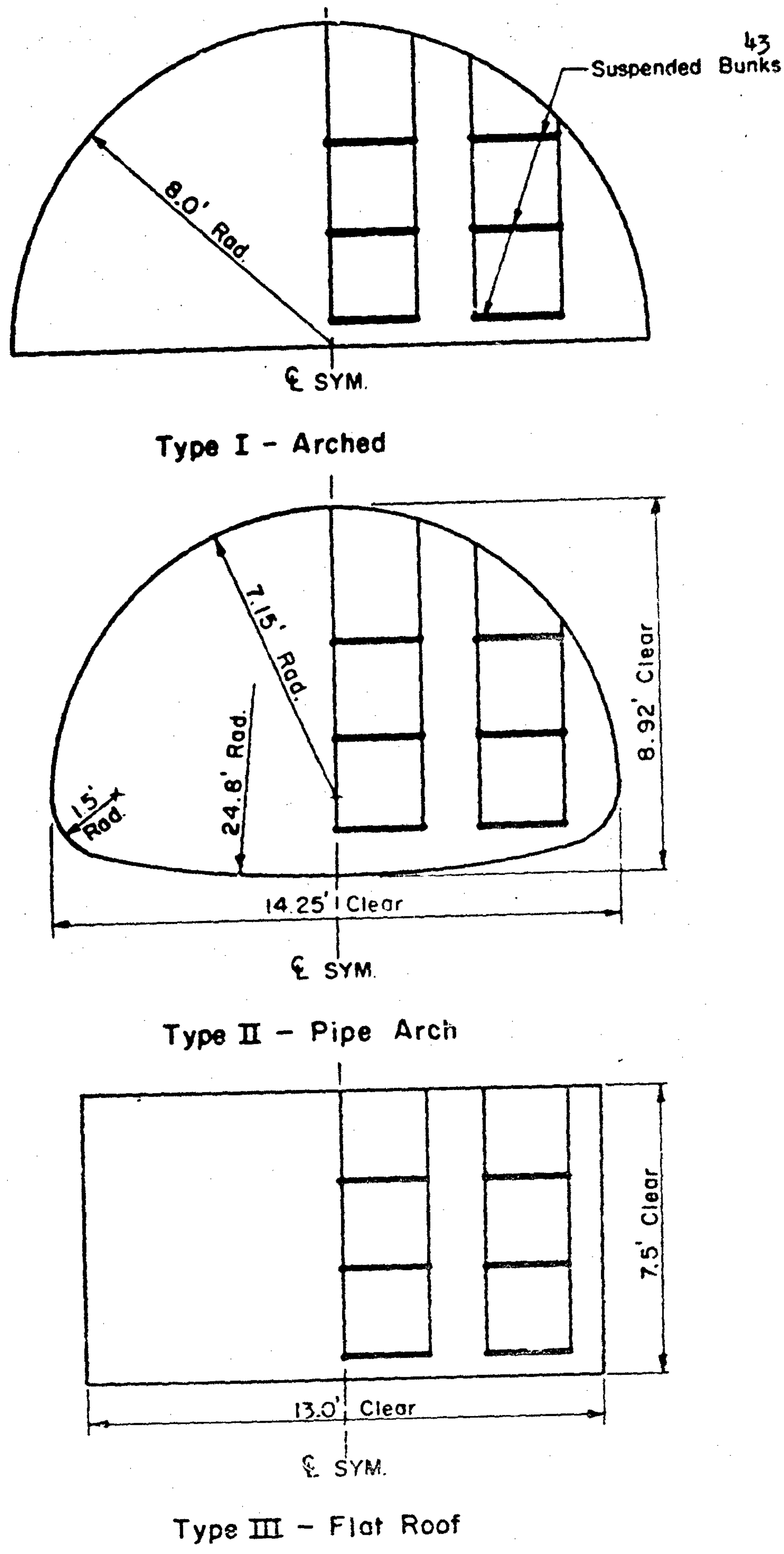
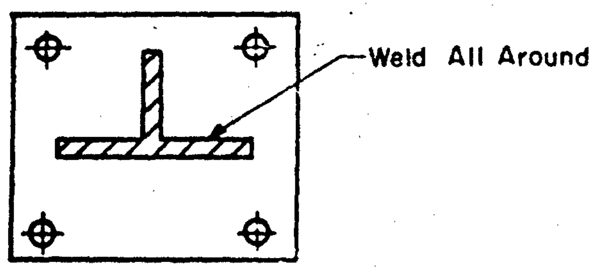
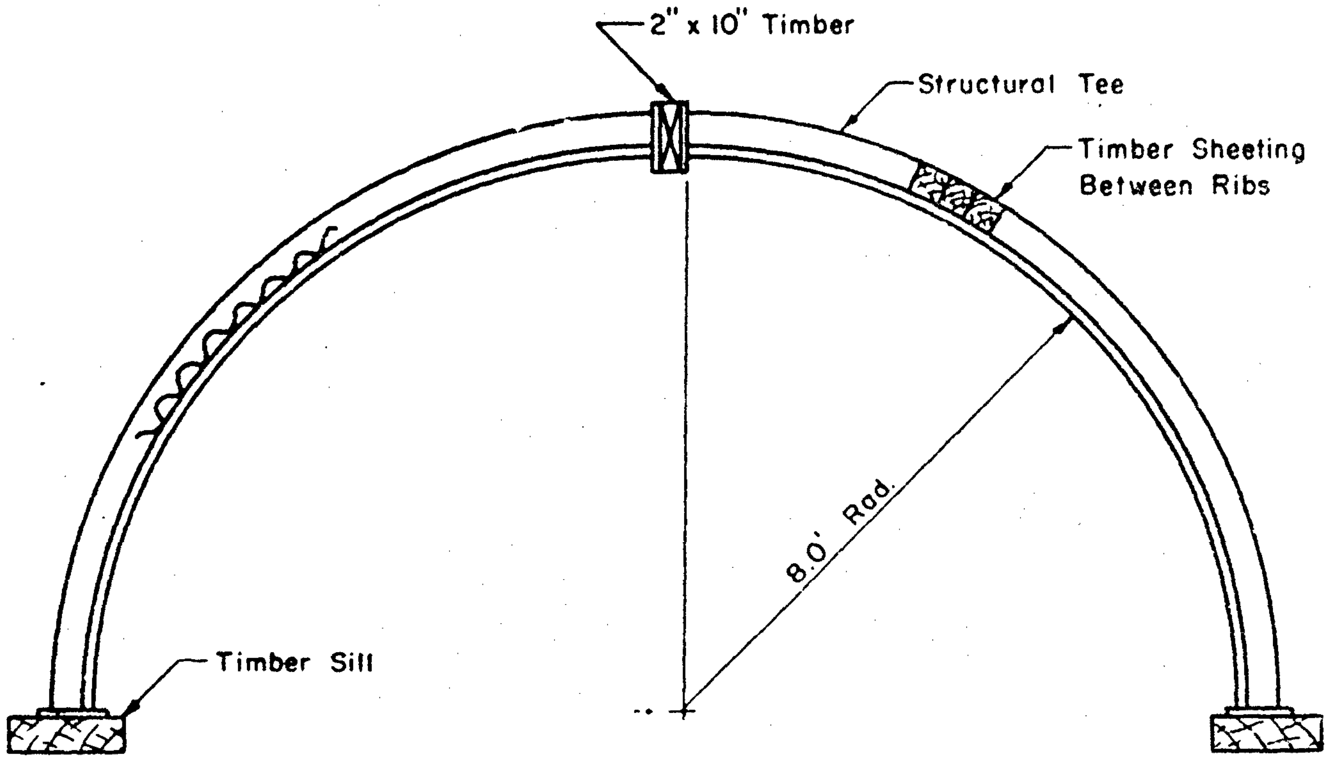


FIG. 3.3 TYPES OF CROSS-SECTIONS



Detail Of Quarter-Circle Rib Ends



Half-Section With Corrugated Plates

Half-Section With Timber Blocks

FIG. 3.4 TYPE I-A CROSS-SECTION

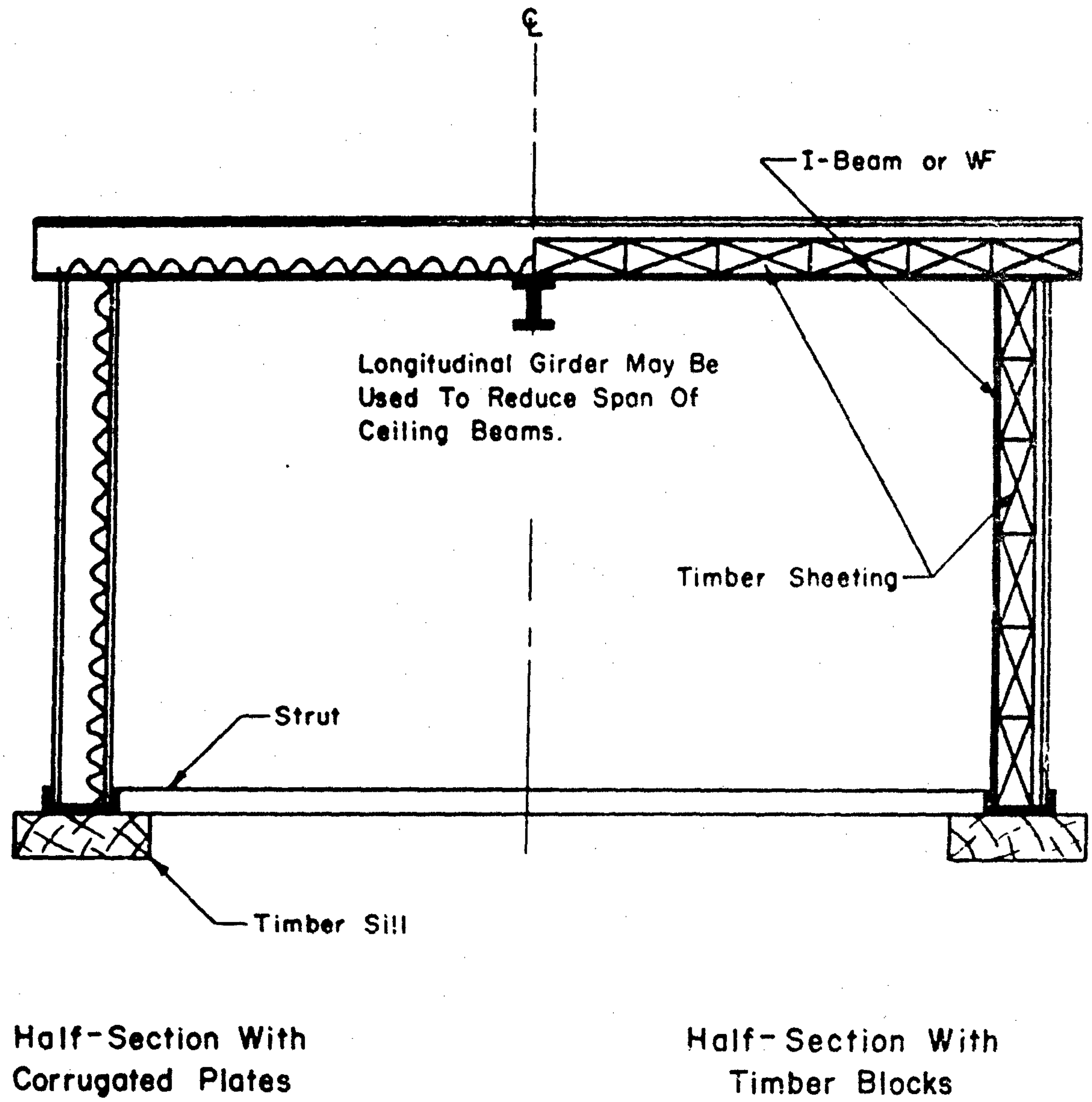


FIG. 3.5 TYPE III CROSS-SECTION

4. DESIGN PARAMETERS

The architectural factors discussed in the preceding chapter provide the means of specifying the type and size of structure which is suited to a particular design situation. With the structural type and size specified it becomes necessary to determine the loads acting on these structural types and the resistance developed by them before proceeding with the design. The precise loading and resistance developed in a particular attack situation generally are complex; those developed in a general attack situation evade definition completely. However, even in this general case, certain conventionalizations may be made and these conventionalized loads and resistances can be specified with relative ease and with confidence. Since in the design problem the attack situation only can be assumed, there is no reasonable justification for using loads and resistances which are more complex than the conventionalized ones. Furthermore, these conventionalizations have a rational basis, they provide relative simplicity in analysis, and they appear neither unsafe nor overly conservative when compared to actual weapon test experience.

In the sections which follow the conventionalized loading functions are discussed first. Since resistance functions in general depend upon the material properties these properties are discussed next in a separate section. Finally, the conventionalized resistance functions are described.

4.1 CONVENTIONALIZED LOADING FUNCTIONS

4.1.1 General Considerations. Buried construction may be accomplished by basically two methods: (1) by locating the structure on the original ground surface and by covering it with extensive earth fill; and (2) by locating the structure below the original ground surface by cut-and-cover means or by tunneling. By either method criteria to define complete burial must be established. Complete burial in this report is defined as that condition under which the net forces acting on the structure are primarily symmetrical. Thus, depths of burial and extent of earch cover must be such that anti-symmetric loads reaching the structure are negligible or that sufficient earth is provided to develop passive pressures of such magnitude to prevent an anti-symmetrical deformation of the over-all structure. For a surface structure covered by an earth mound the mound must be proportioned first to accomplish streamlining to reduce reflections and drag and second to divert these anti-symmetric loads into the original ground away from the structure. For a structure located entirely below the original ground surface the depth of burial must be sufficient at least to develop the necessary passive pressures.

Although there is no direct rational method of computation available as yet, it appears the conditions specified in Ref. (4.1) reasonably establish the criteria for complete burial discussed above. For convenience these criteria are enumerated here. As shown in Fig. 4.1, an earth-mounded surface structure must be covered so that the average depth of cover over the roof is at least equal to one-fourth of the least span; so that, under conditions of minimum cover, the faces of the earth mound are no steeper than 4 horizontally to 1 vertically and so that the distance from the toe

of slope (the intersection of the slope with the original ground) to the nearest point on the structure is at least four times the height of the mound. Region A in Fig. 4.1 is specified to allow for greater depths of cover over the roof when radiation protection or terrain conditions require it. A structure located below the original ground surface is completely buried if the average depth of cover is at least equal to one-quarter of the least span. An exception to the latter specification would be a flat roof structure in which case the top surface of the roof may be made to coincide with the original ground surface. These requirements of complete burial consider only the structural integrity. In some instances requirements for radiation protection may specify larger amounts of earth around the structure.

These larger amounts of earth which might be required for radiation protection generally will not be sufficient to develop an amount of arching within the soil above and adjacent to the structure to reduce significantly the forces acting on the structural elements. On the other hand it may be economical in some instances to provide sufficient depths of burial to develop this phenomenon and to develop significant attenuation of vertical stress in the soil. This would be justified where the cost of structural materials saved would offset the additional cost of excavation. However, arching largely depends upon the deformation characteristics of the structure. These characteristics are not known specifically until the structure is proportioned. As a result, consideration of these effects fall more logically into analysis of the structure proportioned in the preliminary design. This subject is considered in the following chapter. Before leaving this subject, however, it should be noted that

arching and attenuation influence mainly the intensity of the peak pressure; they do not change significantly the basic shape of the forcing function.

In the preceding discussion the attenuation of vertical stress in the soil mass was mentioned. Attenuation must occur as a result of the dispersion of the stress in the soil mass. In this report the attenuation phenomenon has been considered in combination with the foundation motion in an attempt to define the net force acting on the top of a buried structure. The development of this net force is discussed in Appendix F. As noted in this appendix the effects of attenuation of stress with depth and footing motion are insignificant for weapons in approximately the megaton range. Therefore, these attenuation and footing-motion effects may be neglected unless the depth of burial becomes large. Since large depths of burial generally are incompatible with the type of construction emphasized herein, the attenuation and footing-motion effects for these types therefore may be neglected.

Since protective construction perforce utilizes ultimate strength concepts in design, consideration of the factor of safety is important in specifying the forces acting on the structure. Yet, the factor of safety is a rather elusive quantity insofar as the shock loading is concerned. A protective structure must resist the dead loads acting in combination with the shock loads. The dead loads, of course, can be accurately determined, but the shock loads must be determined in a manner which approaches conjecture. The designer first must infer the size of weapon he must protect against. Secondly, he must infer the probable point of detonation of this weapon. From these inferences the most probable overpressure level and shape of the forcing function may be readily determined for the

particular structure under consideration. These necessary assumptions for determining the forcing function can be obtained logically by considering the enemy has the same weapon delivery capability, and his intelligence is sufficient to know as much about the target area, as the designer. These considerations allow the determination of a circular probable error (CEP) and a vulnerability radius from which the overpressure and radiation levels for design may be obtained. As a result the designer chooses a factor of safety when he chooses the degree of hardening required for his structure. Having chosen the degree of hardening required the designer should proportion the structure to resist the maximum shock and dead loads applied simultaneously with no additional factor of safety.

This discussion suggests another factor which might be considered by the designer, that of dispersal versus increased hardening. For the same probability of survival, duplication of units may prove more economical than a single unit which necessarily must be designed to resist much larger forces. From a consideration of structural integrity alone however, construction of two duplicate rectangular structures instead of one becomes economical only when the design overpressure for two structures is approximately one-quarter the design overpressure for a single structure. Similarly, duplication of two arched structures becomes economical only when the design overpressure for two arches is approximately one-half the design overpressure for a single arch. This results from the fact that the depth of individual members varies generally as

the square root of the applied force in flexural members and directly as the applied force in compression members.

4.1.2 Completely Buried Rectangular Structures. Rectangular structures will consist of a horizontal roof with vertical walls. Thus, the forcing function for both the horizontal and vertical members must be defined.

For nuclear weapons the force acting on the roof is the same as that acting on the floor if the floor is the only foundation provided. This force-time loading will be triangular in shape, and may be considered to rise instantaneously to its maximum value followed by a linear decay back to zero. The maximum value of the force may be considered to be the side-on overpressure acting on the surface immediately above the roof. The effective duration of the force usually would be less than the positive phase duration of the overpressure, and it would be defined on the basis of the shape of the overpressure curve. This duration would be such that the area under the actual curve and of the replacement curve would be identical up to the time of maximum response of the member being considered.

The walls also will be subjected to a force-time loading which is triangular in shape. The effective duration of this triangle will be determined in the same manner as for the roof. However, the peak value of the force depends upon the type of soil surrounding the structure, and it equals K times the peak force acting on the roof Ref. (1.2).

K = 0.25 for all cohesionless soils, damp or dry

K = 0.50 for cohesive soils, not saturated

K = 0.75 for cohesive soils of soft consistency

K = 1.00 for all saturated soils where the water table is less than 2 ft. from the surface. Where the water table is more than 20 ft. below the surface use K for the unsaturated condition and interpolate linearly to find K for intermediate positions of the water table.

4.1.3 Completely Buried Arches. Since in a completely buried arch the antisymmetrical deformation is prevented, only a symmetrically applied force is effective in producing deformation. This force-time loading consists of a triangle with a peak value and an effective duration the same as has been discussed for the roof or floor of a rectangular structure. However, this force differs from that on a rectangular structure in that the rise time t_r may be relatively long. This rise or application time results from the passage of the shock through the earth to the structure. From semi-empirical relationships it has been found that the rise time is at least equal to one-half of the transit time for the wave to pass from the ground surface to the arch or

$$t_r = \frac{1}{2} \frac{h_{ave}}{c_v} \tag{4.1}$$

where t_r = rise time, sec.

h_{ave} = average depth of cover over arch, ft.

c_v = seismic velocity, fps

For the case where the load acts transverse to the axis of the arch the rise time will be increased above that given above by the transit time required for the shock to engulf the structure. However, since the point of detonation is unknown this increased time cannot be counted upon.

4.2 MATERIAL PROPERTIES

Because protective construction requires the application of ultimate strength concepts which is a subject not generally covered in most

current specifications, it is desirable to point out the differences which occur for protective design. Since different materials behave differently under dynamic loads, and in fact differently under static loads in some cases, the properties of basic materials are discussed under separate headings.

4.2.1 Metals. Under rapidly applied loads metals in general develop a yield strength which is higher than the similar strength developed under statically applied loads. The amount of the increase depends upon the strain rate induced in the material. For the strain rates likely for materials subjected to the effects of nuclear weapons an increase in yield strength 25 percent above the normally specified value would be reasonable for design purposes. However, since some materials may be relatively brittle the dynamic yield strength never should be assumed greater than 90 percent of the ultimate tensile strength. One related phenomenon also should be mentioned. This phenomenon is characteristic of metals, such as ASTM-A7 steel, which possess a flat yield region. These metals develop an increased yield strength, and at the same time they may retain this elevated strength without plastic deformation for a measurable amount of time; a characteristic referred to commonly as time-delay of yield. A time-delay of yield, if it develops, provides an increased bonus for dynamic loads since the forcing function may decay sufficiently during the time delay that the metal will never yield. From the standpoint of design this bonus is very desirable although it should not be counted upon.

The above discussion gives a means of specifying the yield strength for use in the formulae of current specifications. For strength governed by shear 60 percent of the dynamic yield strength in tension

should be used. For strength governed by buckling considerations, the design stress used must not exceed the yield strength if an arch is completely buried. If a semi-circular barrel arch with hinged supports is completely exposed the maximum side-on overpressure as limited by buckling considerations is

$$p_{cr} = \frac{3EI}{r'^3} \tag{4.2}$$

where p_{cr} = maximum overpressure specified by buckling

E = modulus of elasticity

I = moment of inertia per unit width of the arch

r' = mean radius of arch.

On the basis of empirical results buckling may be neglected for the completely buried arch.

In addition to allowable stresses a shape factor F may be applied to flexural members of metal. This shape factor depends upon the configuration of the cross-section. It may be determined by evaluating the moment resistance developed by assuming a uniform stress block exists above and below the neutral axis of the section as compared to the moment resistance corresponding to the conventional linear distribution of stress. For rectangular cross-sections F = 1.5 while for wide flange and standard beams F varies from approximately 1.05 to 1.2. Thus, for wide flange and standard beams F may be taken at an average value such as 1.1. In general the yield moment developed in flexure is

$$M_y = F \frac{f_y I}{c} \tag{4.3}$$

where M_y = the effective yield moment used in the elasto-plastic replacement of the actual resistance function.

Finally the designer may encounter some cases where a metal member is subjected simultaneously to an axial load and a moment. The ultimate resistance under such a condition may be determined from an interaction diagram. Such a diagram must be developed for the particular section being considered. This development may be accomplished by assuming successive positions of the neutral axis and finding the axial force and moment combination which corresponds to each assumed position of the neutral axis.

4.2.2 Timber. Timber behaves rather unusually under instantaneously applied loads in that it develops a strength two or more times the usual static strength under such loads even if the load duration is one or two seconds. At the other extreme of loading timber has a reduced strength under long continued load of constant magnitude. The allowable stresses specified for the design of timber under conventional loads are reduced to account for the long continued load effect; however, no allowance is made in the conventionally specified stresses for the dynamic effect. To obtain the design strengths comparable to all of the controlling stresses given in current timber specifications the allowable stresses may be multiplied by four if the timber specified is of stress-grade. If the timber specified is not of stress-grade the allowable stresses given in current timber specifications may be multiplied by two to determine the design strengths for shock loads.

Because timber is a relatively brittle material it must be proportioned for nearly elastic behavior under the dynamic loads. Also because of its brittle nature the redistribution of stress implied by the form factor in the preceding section cannot occur. Therefore, flexural behavior is governed by the conventional formulae without modification except for

the increased allowable stresses. For the same reason, timber members subjected simultaneously to axial force and moment should be designed in accordance with the following equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1 \qquad (4.4)$$

where f_a = actual stress induced by axial force

f_b = actual stress induced by moment

F_a = allowable stress for axial force alone

F_b = allowable stress for moment alone

4.2.3 Precast Reinforced Concrete. Because the emphasis of this report is placed upon the analysis and design of a shelter which might be fabricated in the field by an engineer platoon with their normal component of equipment, the use of reinforced concrete is restricted to precast elements of very limited weight. At most such elements could be no larger than would be required to span short distances between ribs of an arch or between beams of a rectangular structure. Thus, these members would be restricted in their behavior to act as beams and probably only as simply-supported beams. Consequently, only simply-supported precast reinforced-concrete beams are considered in the following discussion.

The resistance developed by the beams under consideration would be controlled by one of five possible modes of failure. These modes are: flexure, diagonal tension, shear-compression, pure shear, and bond. An amount of web reinforcement can be provided which is sufficient to preclude diagonal tension and shear-compression failure and in the following discussion this procedure has been assumed. However, to delineate between the case where web reinforcement is not required and the case where such

reinforcement is required the diagonal tension resistance is specified, but it is used only for this purpose and it is not to be used in the determination of resistance. The significance of the diagonal tension resistance is shown in Fig. 4.2. For resistances less than those corresponding to Point 'A' in the figure flexural behavior is assured. For resistances larger than those corresponding to Point 'A' web reinforcement <u>may</u> be required. It is only required if φ_v in Eq. (4.7) is a positive value.

The flexural resistance may be specified in terms of the width of the member b, the percent of longitudinal tension reinforcement φ which should not be less than 0.25 nor more than 1.5 percent, the dynamic yield strength of the steel f_y, and the depth d to span L ratio as

$$r_f = 0.0716\ \varphi f_y b\ \left(\frac{d}{L}\right)^2 \qquad (4.5)$$

Web reinforcement <u>may</u> need to be supplied if the required resistance as indicated in Fig. 4.2 exceeds

$$r_{dt} = 3.5b\ \left(\frac{d}{L}\right)\ \sqrt{f'_c} \qquad (4.6)$$

where f'_c = standard cylinder strength of concrete in psi

The amount of web reinforcement φ_v required if the resistance needed exceeds r_{dt} may be specified as below where C is the ratio of the percentage of longitudinal compression reinforcement to the longitudinal tensile reinforcement and f_v is the yield strength of the web reinforcement.

$$\varphi_v = 34\ (2 + C)\ \frac{f_y}{f_v} \sqrt{\frac{\varphi}{f'_c}} - \frac{5 \times 10^4 \text{ psi}}{f_v} \qquad (4.7)$$

The parameter C is determined by the amount of reinforcement required to resist the elastic rebound of the beam. Even neglecting the possibility of rebound it is desirable to use at least a nominal amount of compression reinforcement since it increases the ductility of the member. Generally for long duration loading rebound resistance requires a resistance which is on the order of one-fourth the principal resistance of the beam. Web reinforcement, where required, must consist of vertical stirrups because of the rebound problem. Also when web reinforcement is required, no less than 0.5 percent should be used.

The resistance to pure shear is defined by

$$r_{sp} = \frac{1}{3} f'_c b \left(\frac{d}{L}\right) \tag{4.8}$$

Bond failure is prevented if deformed bars are used which have a size such that twice the allowable bond stresses given in the ACI Building Code, Ref. (4.2), are not exceeded.

4.3 CONVENTIONALIZED RESISTANCE FUNCTIONS

The ultimate resistance of any member may be characterized reasonably as being bilinearly elasto-plastic. Therefore, the entire resistance can be defined by specifying three parameters: (1) the yield resistance r; (2) the natural period of vibration, T; and (3) the ductility ratio μ which is the ratio of the maximum deflection x_m to the effective yield deflection x_y. In some instances it is desirable to know the yield deflection. Therefore, the three parameters listed above and the yield deflection are defined below for the types of construction considered in this report.

4.3.1 Rectangular Structures. The roof and floor, if a floor is required, of a rectangular structure are subjected primarily to flexure and secondarily to axial force. Generally, however, the axial force is small compared to the flexure and without serious error its presence may be ignored. As a result the yield resistance r is determined from the flexural resistance of the elements forming the roof or floor. The natural period of vibration T of these elements alone is given by

$$T = \lambda L^2 \sqrt{\frac{m}{EI}} \tag{4.9}$$

where λ = 0.636 for simply supported elements

= 0.318 for homogeneous prismatic members continuous over several supports

= 0.424 for homogeneous prismatic members fixed at one end and simply supported at the other

L = span of member

m = mass per unit length of member

E = modulus of elasticity of material

I = moment of inertia of cross-section (of the transformed section in reinforced concrete).

Where there is earth over the member T must be modified to

$$T' = T \sqrt{\frac{m'}{m}} \tag{4.10}$$

where m' is the mass of the soil and the element but the depth of the soil should not be taken larger than the span L. For the three basic materials considered in this report the value of the radical ($\sqrt{\frac{m}{EI}}$) in Eq. (4.9) may be replaced by

$$\sqrt{\frac{m}{EI}} = \frac{1}{17{,}000 \text{ (ft. per sec.)} R} \cong \frac{1}{6{,}800 \text{ (ft. per sec.)} d}$$ for

steel sections where R is the radius of gyration for a general cross-section and d is the depth of a wide-flange or standard beam.

$$\sqrt{\frac{m}{EI}} = \frac{1}{3{,}150 \text{ (ft. per sec.)} d}$$ for rectangular timber sections

$$\sqrt{\frac{m}{EI}} = \frac{1}{2{,}250 \text{ (ft. per sec.)} d\sqrt{\varphi}}$$ for rectangular reinforced concrete sections.

The ductility ratio μ must be chosen for any design. Choice of this value depends upon the amount of damage which might be allowed under a given situation. Since a factor of safety is implied by the choice of the overpressure for which the design is to be made and since for the types of construction emphasized in this report more than a single attack causing maximum conditions would be unlikely, it is reasonable to assume a relatively large value for the ductility ratio. For steel and reinforced concrete members a value of five is recommended for μ. For timber, because of its inherent brittle behavior, a ductility ratio no greater than 1.5 is recommended. Yield deflections of simply-supported flexural members may be approximated by the following

$$\frac{x_y}{L} = \frac{L}{\lambda' d} \tag{4.11}$$

where $\lambda' = 3000$ for steel

$\lambda' = 800$ for timber

$\lambda' = 4000$ for reinforced concrete

In this equation, x_y and L on the left must be in the same units, and L and d on the right must be in the same units, which may differ from the units used on the left side.

For a beam fixed at both ends the effective yield deflection is approximately one-half that of a simply-supported beam of the same dimensions and span. Similarly the effective yield deflection of a beam fixed at one end and simply supported at the other is approximately two-thirds that of the equivalent simply-supported beam.

Exterior walls of rectangular structures must support the roof as well as resist the lateral loading. The axial force on these elements generally is comparable to the flexural force; therefore, this axial force cannot be neglected. However, the axial force in the wall results from the support of the roof slab. This reaction on the slab will have a rise time which is much larger than the natural period of longitudinal vibration of the wall; thus it may be treated as a statically applied load. On the other hand the shock loading causes flexure in the wall which must be considered as dynamically applied. The resulting resistance of the wall must be determined from an interaction diagram as discussed briefly in Section 4.2.

Interior columns and beams of a rectangular structure support the load which corresponds to the yield resistance of the tributary elements. These loads may be considered as statically applied for the same reason as mentioned with regard to the axial force on exterior walls.

4.3.2 Completely Buried Barrel Arches. A completely buried barrel arch develops its resistance as a hoop compression. The maximum thrust S developed under this condition is

$$S = r\,br'$$

where r = yield resistance corresponding to the peak vertical stress acting at the crown of the arch,

b = width of the section of arch considered,

r' = mean radius of the arch.

The yield resistance then equals the thrust divided by the area of the arch cross-section. Empirical results summarized in Appendix F, indicate that buckling may be neglected in completely buried arches. It is important to insure the joints in a barrel arch have sufficient strength to develop the strength of the main arch. This may be accomplished easily by butting the two sections of the main arch together and using a single or double splice plate. A single splice plate on the interior of the rib generally will suffice.

The natural period of vibration corresponding to this hoop compression in the barrel or ribbed arch would be

$$T = \frac{r'}{2700 \text{ ft. per sec.}} \quad \text{for steel sections} \tag{4.13}$$

To account for the earth cover T must be modified in accordance with Eq. (4.10) but the depth of soil should not be assumed greater than one-fourth the span. Only steel sections are considered here because of the limitations imposed by the equipment available to an engineer company.

A ductility ratio of three is recommended for the design of such sections. Because of the mode of deformation involved, the yield deflection of a barrel or ribbed arch has most meaning when it is defined in terms of the change in radius of the arch; i.e.,

$$x_y = 0.0016\, r' \tag{4.14}$$

In Eq. (4.14) the deflection corresponds to a deformation occurring radially inward.

4.3.3 Completely Buried Ribbed Arches. Because of the limitations of equipment available a ribbed arch will consist of steel ribs supporting as simple beams timber, precast concrete, or corrugated metal elements. The design of the supported elements would be accomplished in the same manner as discussed for the roof and floor of a rectangular structure in Section 4.3.1. The design of the ribs may be accomplished by the methods presented in the section immediately above.

From considerations of weight and ease of construction it appears that a shelter consisting of inverted structural steel tees acting as ribs of an arch supporting timber blocks is the preferable configuration for the structure housing personnel in a forward area. For this type of construction the tees must be designed to carry the entire hoop compression if the wood blocks are not chamfered such that each block bears uniformly against its neighbors. On the other hand if the timber blocks are chamfered such that they bear against one another throughout their length, composite action of the timber blocks and the steel ribs may be used in the design to resist the hoop compression. As indicated in the design calculations presented in Appendix C, a shelter to provide protection against an overpressure of 100 psi can be porportioned using relatively light individual members neglecting any composite action. Since some composite action must occur even if the blocks are not chamfered, the structure designed in the example will doubtless resist somewhat more than the 100 psi for which it was designed. Yet, by chamfering the blocks and reducing the spacing between the ribs, the proposed structure can withstand an overpressure as high as 300 psi.

The slight over-design of the standard structure mentioned above (designed for 100 psi as required by the criteria set by WES) is considered desirable because, as implied here, by supplying additional steel ribs and a cross-cut saw, this standard structure can be tailored to resist overpressures as high as 300 psi should the need arise. The necessary modifications are shown in Appendix H.

4.3.4 Foundations. Since the integrity of the foundations of a conventional structure determines the integrity of the structure, the foundations of such structures normally are designed more conservatively than the superstructure. It is desirable here therefore to compare the integrity of conventional structures with the integrity of protective structures. For a conventional building, integrity requires no differential settlement over long periods of time, preferably no settlement of the over-all foundation, minimum disturbance to surrounding buildings, and related considerations. For a protective structure, integrity requires only that the structure protect the occupants from the blast and associated effects for relatively short periods of time. Thus design of foundations for protective shelters can be approached somewhat differently from the conventional case.

For the configuration considered herein differential settlement can be allowed so long as the exit is not completely closed off and so long as the blocks between the ribs of the arch remain in place. Considering the shelter designed herein indicates that a differential settlement of at least 0.6 ft. must occur in a length of 3 ft. before timber blocks between the ribs will fall. Also, as long as the exit is not completely closed off, considerable over-all settlement of the structure can be permitted. The only problem remaining from the conventional design, therefore,

is that of disturbing surrounding structures. For a single shelter in a forward area this problem need not be considered. The effect of multiple structures is discussed in Appendix G.

Since, by conventional standards, large differential and over-all settlements can be allowed for the structure herein, the problem of the design of the foundations becomes a simple one. Recognizing that the shock loading has a longer duration but a much lower force generally than the output of a pile driver indicates that the displacement of any foundation should not be significantly different from the displacement of a pile as a result of a single blow of a pile driver. Thus, it is expected that any foundation for the shelter will undergo a very small displacement, and the results of the weapon effect tests indicate this is so. As a result, it is only necessary to guard against large relative displacements, and the sills for the ribbed arch considered herein are designed to give full continuity throughout their length. These are flexural members primarily and they are designed on the basis of the recommendations in Section 4.3.1.

4.3.5 End Bulkheads. As discussed in Section 3.2.5 several different bulkhead configurations were considered. Three of the configurations were considered in detail. These three different bulkheads all assumed the same type of bulkhead, i.e., vertical wide flange posts with timber blocks spanning between the vertical posts. In each case the top of the vertical bulkhead posts rested against the end structural arch rib of the main shelter. The three differed only in the means of resisting the lower reaction of the bulkhead posts. The three types considered in detail were

(1) A heavy wide flange beam spanning between the ends of the sill.

(2) A horizontal or nearly horizontal steel arch fastened to the ends of the sill with the lower end of the vertical bulkhead posts tied to the arch by cables.

(3) A prefabricated truss spanning between the ends of the sill.

Preliminary designs indicated that the horizontal steel arch was the most feasible means of supporting the lower reaction of the vertical posts. The major advantage of this type of support was that since the arch was primarily in compression and had but a minimum of bending moment its cross section could be used very efficiently. The moment could be held to a minimum by crossing the cable ties carrying the low reaction of the vertical posts to the arch itself. The ties were positioned to eliminate moment at the crown and to cause the pressure line to follow the neutral axis as closely as possible. The arch, however, was found to be unsatisfactory when it was considered that erection procedures necessitated some means of supporting the vertical posts laterally until the blocks could be placed and the backfill completed. The addition of a cross beam to temporarily support the vertical beams increased the weight of the overall assembly to the extent that it was no longer feasible.

A single wide flange cross beam spanning between the two sills provides all the desirable characteristics required of the supporting systems with the exception that the weight of the beam is much too heavy to be placed manually.

The welded truss as finally selected as the most desirable means of supporting the lower ends of the vertical posts actually is a compromise between an arch and a cross beam. The depth of a truss was selected such

that there would be no shear in the panel for symmetrical loadings. Thus for all practical purposes the upper chord becomes an arch while the lower chord serves only as a tie. Diagonals were added, however, to provide for limited antisymmetrical loadings.

4.3.6 Vertically Oriented Entrance and Ventilation Structures. Since the design of the entrance and ventilation structures will be accomplished in a similar way, both types of structure are considered in this section. At some distance below the surface of the ground the deformation of the hollow cylinder used for either of these structures must be uniform all around the cylinder. This results from the fact that any tendency toward non-uniform deformation will mobilize resistance within the soil preventing these non-uniformities from developing. The depth at which uniform deformation must occur cannot be calculated exactly by methods available at this time. However, it seems reasonable to assume that uniform deformation must occur below a depth equal to one-half of the diameter of the cylinder. Because these cylinders are relatively short the variation in lateral soil pressure along the length may be neglected, and only the lateral shock pressure as defined in Section 4.1.2 for vertical walls of rectangular structures may be considered as acting uniformly around the cylinder below a depth of one-half the diameter of the cylinder. Above this depth the lateral pressure of the soil also may be neglected, but, to account for non-uniform deformation, two components of shock loading are suggested. One component is equal to one-half of the side-on overpressure acting uniformly around the periphery of the cylinder; the other component has a peak amplitude equal to one-half of the surface overpressure

acting sinusoidally around the periphery assuming that two complete sine waves develop around the circumference of the cylinder. These components of loading are shown in Fig. 4.3.

The uniform components of loading on the cylinder produce a thrust in the shell defined by Eq. (4.12). The sinusoidally varying load produces a maximum bending moment per unit of height of

$$M_{max} = \frac{1}{3} p_{so} r'^2 \tag{4.15}$$

With the thrust and bending moment defined the cylinder may be proportioned, with slight conservatism, by use of Eq. (4.4).

To prevent particles of radioactive fallout from being sucked into the ventilation structure, it is necessary for these structures to project some distance above the surface of the ground. It is recommended that this projection be made 2 ft. With this projection and with no concrete or soil supporting it the ventilation tube will be subjected primarily to drag loading. It must resist this loading in flexure. Using a drag coefficient of 0.6, which appears valid for cylindrical shapes, gives the following total drag force.

$$F_d = (14 \text{ lb.})\left(\frac{D}{1 \text{ in.}}\right)\left(\frac{p_d}{1 \text{ psi}}\right) \tag{4.16}$$

and by assuming that the cylinder is fixed against rotation at a distance of 4 ft. below the top of the tube, the maximum moment produced by drag is

$$M_d = (500 \text{ in.-lb.})\left(\frac{D}{1 \text{ in.}}\right)\left(\frac{p_d}{1 \text{ psi}}\right) \tag{4.17}$$

where F_d = drag force

D = diameter of cylinder

p_{so} = side-on overpressure at surface, psi

p_a = ambient pressure ahead of shock = 14.7 psi at sea level

p_d = drag pressure, psi

$= \dfrac{2.5\, p_{so}^2}{7p_a + p_{so}}$ for ideal conditions.

M_d = maximum bending moment produced by drag.

Thus, to resist this drag force the section modulus of the unsupported cylinder must be such that the yield stress of the material is not exceeded.

So that there will be a minimum of field sorting of parts, and for ease of erection, it is recommended that the thickness of the vertical cylinders discussed above be held constant. Thus, the thickness would be determined from the condition of loading which required the greatest thickness.

Design of the hatch at the surface of the ground to cover the entrance depends upon the structural shape selected. It is recommended that the hatch be in the form of a segment of a spherical dome so that it may be as light as possible. The resistance of a dome in the configuration discussed in Chapter 3 is

$$r = \frac{2\sigma_y t}{r'} \qquad (4.18)$$

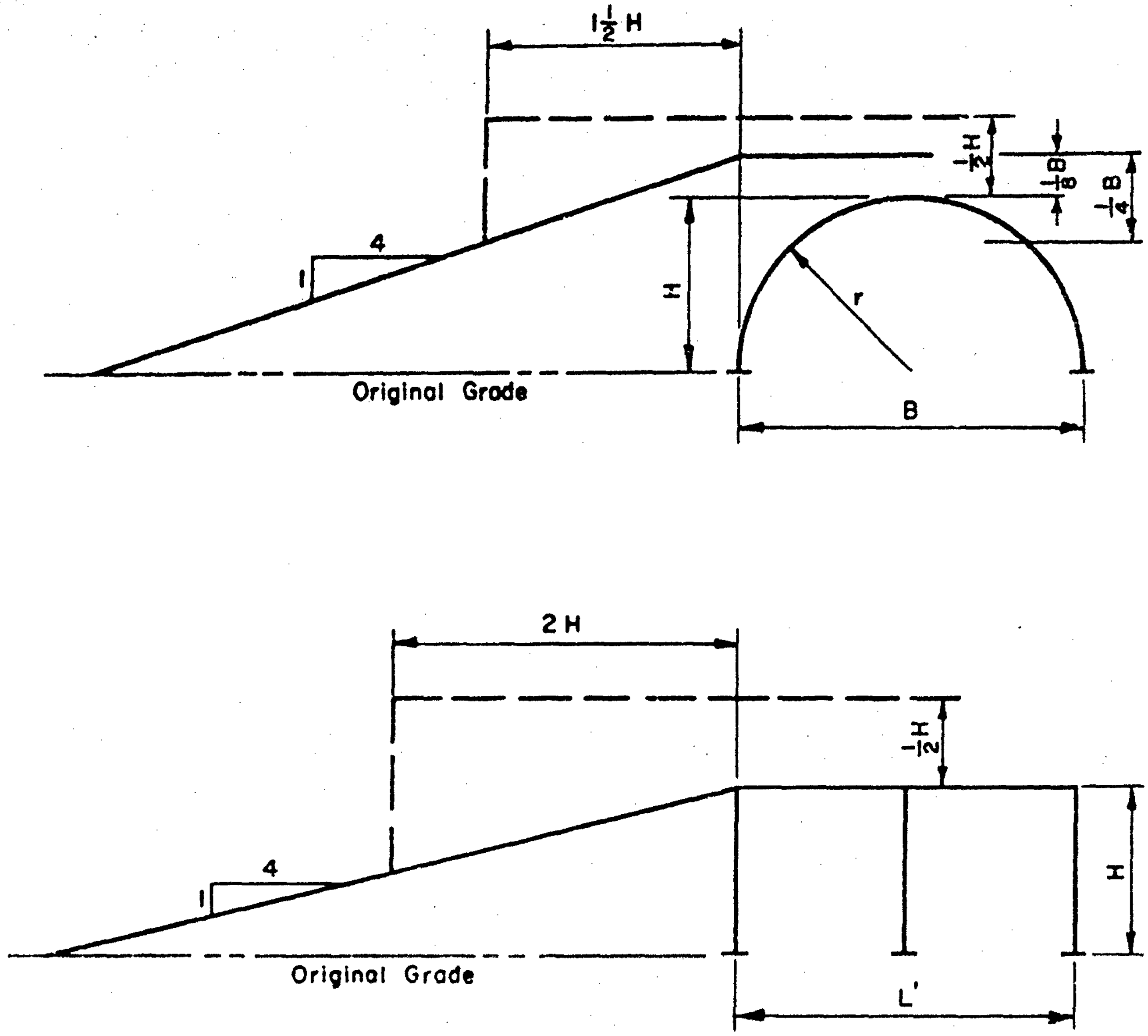
where r = resistance required for the shock loading

σ_y = yield stress of the material

t = thickness of the dome

r' = the radius of the major circle of the dome

Because of the geometrical shapes involved in the entrance and ventilation structures, it will be difficult to modify these structures in the field to provide protection against overpressures larger than those considered in the standard design. Since the main structure requires only relatively minor field modification to provide an increased level of protection, it may be desirable to provide entranceways and ventilators designed to resist overpressures as high as the standard structure may be made to withstand as a result of field modification.



——————Boundaries of Minimum Cover.

— — — Boundaries of Region A. Cover in Region A may result from topographic conditions. In Region A the maximum slope permitted is 1 on 2.

FIG. 4.1 RECOMMENDED DEPTHS OF COVER FOR FULL BURIAL

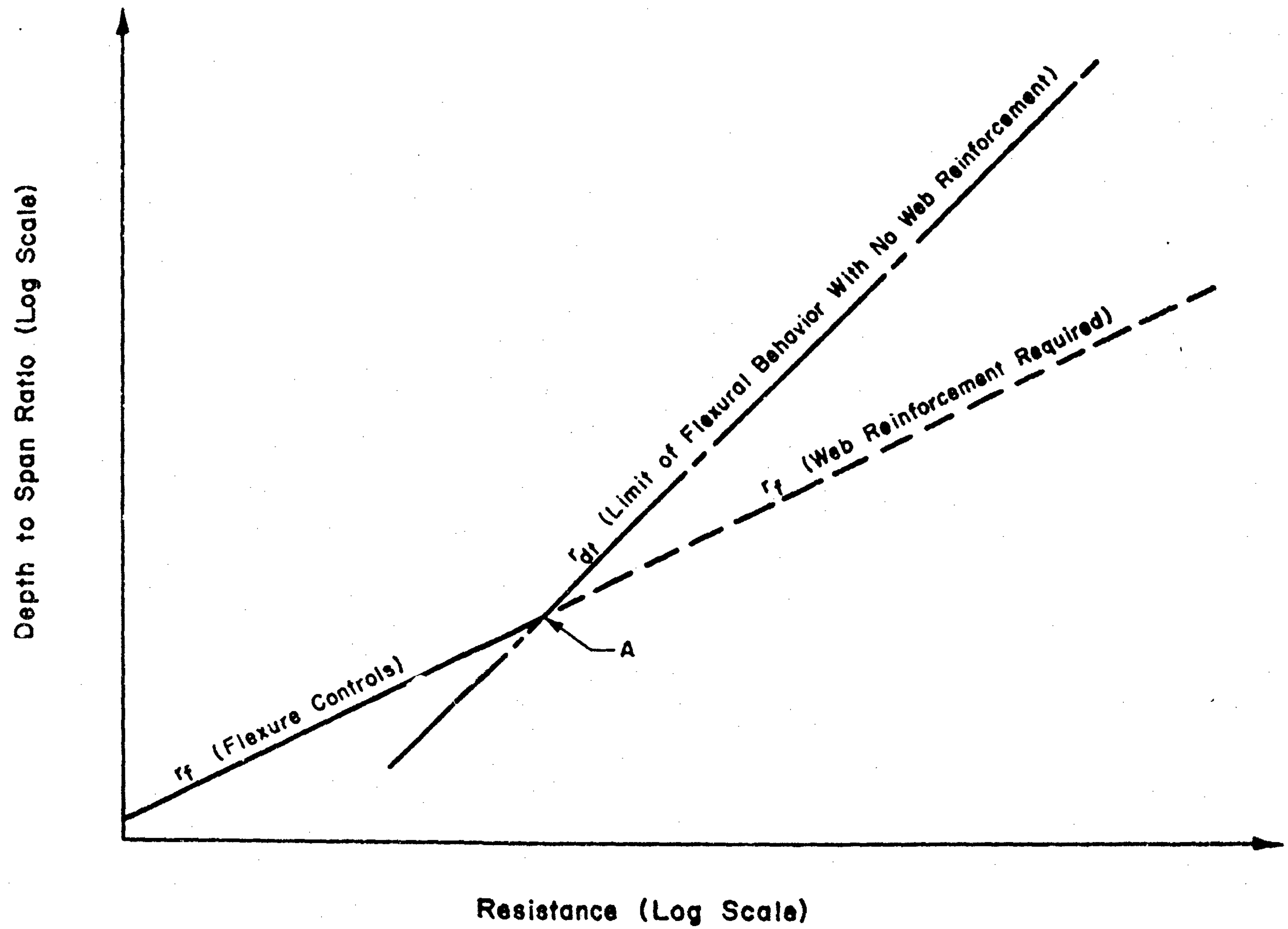
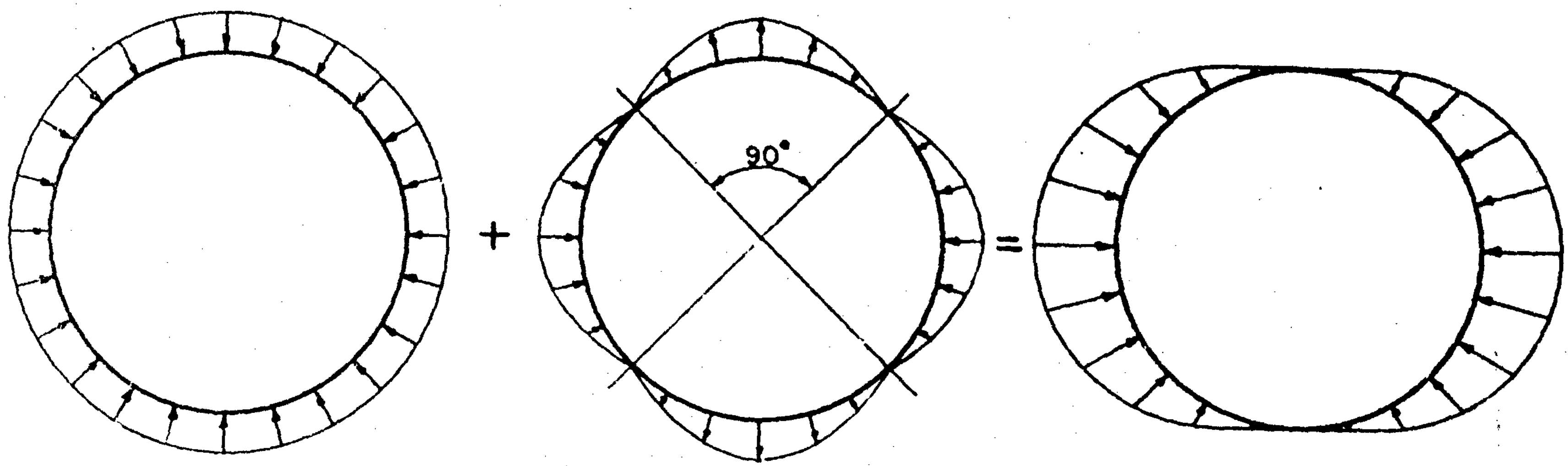


FIG. 4.2 CRITERIA DEFINING MODES OF FAILURE IN REINFORCED CONCRETE.



Uniform Pressure Of $\frac{p_{so}}{2}$ + Sinusoidal Pressure Of Amplitude $\frac{p_{so}}{2}$ = Net Pressure

FIG. 4.3 SUGGESTED COMPONENTS OF LOADING ON UPPER PORTION OF ENTRANCE AND VENTILATING STRUCTURES.

5. DESIGN SUMMARY

The architectural details and design parameters necessary to proportion a structure have been discussed in the preceding chapters. This chapter deals with the actual design of the proposed structure. Included are a discussion of the preliminary proportioning of the structural components, a method of analyzing dynamically the resulting structure and comments pertinent to the actual structural design calculations (Appendix C). Detailed plans for the proposed structure are included in Appendix J.

5.1 PRELIMINARY DESIGN

If an attack should come it is probable that nuclear weapons of relatively high yield would be used. The effective duration of the loading would be long compared to the natural period of elements of the structure for this type of weapon. Even if the duration were not relatively long this assumption errs on the side of safety. This assumption and the fact that in a protective design some plastic behavior must occur allows the designer to proportion initially the structural elements to resist pressures consistent with the maximum side-on overpressure applied at the earth's surface above the structure in combination with the dead load as a static load. One exception to this general procedure which may be followed involves the design of timber blocks. Since timber is inherently more brittle than the other structural materials considered it is desirable to multiply the shock loading by 1.5 in the preliminary design to account for its smaller ductility. Dynamic yield stresses in the material should be assumed in this proportioning. The resulting structure may be analyzed by the method presented in the next section.

5.2 ANALYSIS

The static resistance required in an element subjected to shock loading, where the rise time of the loading is at least as long as twice the natural period of vibration of the structural element, is equal to the peak value of the shock loading plus the dead load. As stated in the preceding chapter a finite rise time should be considered possible only on arched structures. Therefore, the rise time of the loading on the arch rib must be computed. If this is greater than twice the natural period of vibration, the arch rib proportioned in the preliminary design is adequate.

When the ratio of rise time to period is less than two for an arch rib or for all other types of structural elements the equivalent static resistance r necessary for shock loads must be determined from

$$p_m/r = \frac{T}{\pi t_d}\sqrt{2\mu - 1} + \frac{1 - \frac{1}{2\mu}}{1 + \frac{2T}{\pi t_d}} \qquad (5.1)$$

Because in most cases t_d will be much larger than T, the first term of Eq. (5.1) generally will be much smaller than the second; frequently this first term may be neglected, and the denominator of the second term taken as unity. The resistance of the element obtained in the preliminary design must be adjusted to equal the sum of the resistance obtained from Eq. (5.1) and the dead load.

5.3 COMMENTARY ON DESIGN

The actual structural design calculations for the proposed structure are included as Appendix C. These calculations follow the preliminary design procedure of Section 5.1. The design for the basic structure was not analyzed completely in accordance with Section 5.2, due to the fact that

the periods of vibration of the individual members are at the most a few tens of milliseconds. For weapons of current operational size (MT range) the effective duration of the overpressure is at least 100 msec. Thus, the condition of relatively long duration of the loading mentioned in Section 5.1 exists for the structure considered here. Since the effective positive phase of smaller weapons is significantly less than 100 msec., the proposed structure is more than adequate for these weapons. An abbreviated dynamic analysis is included in Appendix H.

It should be noted that the values of the ductility ratios suggested in Chapter 4 were chosen conservatively to provide a larger measure of safety in the event of multiple attacks; ductility ratios greater than those suggested are possible without collapse under a single loading. Therefore, for a single loading the design described in the following will resist the design overpressure of 100 psi. For multiple attacks, it is improbable that each attack will produce the maximum overpressure, and it is not considered necessary to alter the design presented in order to resist a multiple attack.

A compressive stress of 3000 psi, perpendicular to the grain, was used to determine the size of the bearing plate at the top and bottom of each main rib. This will result in some crushing of the timbers, but this fact will be more helpful than harmful since it will tend to reduce the shock loading. A 2" x 12" timber is used between the two top bearing plates of the arch ribs. This will give a deformable bearing surface between the plates and will serve to speed the erection of the ribs. Machine bolts are used in the top while lag screws are used to connect the bottom bearing plates to the foundation sills. The sills are pre-bored to accept the lag screws thereby facilitating the rapid positioning of the arch ribs.

The computation for the number and size of corrugated plates is included principally to show why the corrugated plates are not recommended in the standard design considered herein.

A stress of 6000 psi has been used in computing the thickness of timber lagging so that under both bending and axial stresses the combined stresses will satisfy Eq. (4.4).

As previously discussed, the actual width of the foundation sill was determined more by the width required by the bearing plates than from an allowable soil pressure. The net soil pressure has been computed in order that the moments in the sill may be estimated. While the computed soil bearing pressure is very high by conventional standards or practice, it is not unreasonable for dynamic loadings.

The sill has been designed as a continuous beam of constant stiffness. The maximum bending moment for uniform upward soil pressure occurs under the first arch rib, but the actual moment, owing to the redistribution of soil pressure with deflection, is self limiting. Hence, the design moment used is somewhat less than the moment under the cantilever stub at the end arch rib, and somewhat greater than the moment under the interior ribs, since moments will be made more uniform with relative deflections of the points of support of the ribs.

The calculations pertaining to the end bulkhead compare the relative weights of a cross-beam and a truss to transmit the lower reactions of the vertical bulkhead posts. The truss weighs about half as much as the cross-beam. The truss is designed to have no shear in the panels for symmetrical loadings. However, light diagonals are provided in the detail drawings.

The preliminary design for the vertical entrance pipe was a No. 1 ga. multiplate section. However, it is impossible to form this into a 32 in. or 36 in. diameter (60 in. is the minimum diameter that can be fabricated.) and retain the corrugations. A cylinder formed from 5/8 in. plate steel would weigh approximately 600 lbs. and would be difficult to handle. Although the double corrugated sections, with concrete in the annular ring, weigh approximately the same, they are more easily placed in position with subsequent casting of the concrete. Since the corrugated sections serve as the forms and the concrete is not required to carry any dead load, the use of concrete in this element will not delay the construction.

To withstand the 100 psi side-on overpressure, a thickness of 0.03 in. only is required for the hatch cover. However, as this is too thin to weld to the hinges and to the circumferential angle and as it is susceptible to distortion while being transported, installed, or in use, a thickness of 1/4" was arbitrarily selected. Furthermore, standard heads are available with the requisite major and minor radii and thickness. These standard heads require only the removal of the rim to be used as the hatch cover.

The passageway leading from the entrance tube into the structure consists of steel frames fabricated from steel beams that support walls of timber sheeting and a roof of prefabricated steel covers. To reduce the weight of the frames, temporary braces are provided at mid-height. The braces are required only during periods of alert. The entrance tube is attached directly to the passageway by removing one of the steel covers.

Utility structures are similar to the passageway except that four steel frames instead of five are provided and also a crawlway is provided as an access through the earth bulkheads which may be installed. These earth bulkheads may be required to provide a radiation barrier around any CBR unit or a decontamination station installed in the utility structure or passageway. Since a passageway is always required while utility structures are optional, the end bulkhead for the passageway or utility structure is included as part of the passageway kit.

A wide range of sizes for the ventilation ducts is given on the plans because of the unknown conditions which might prevail. For example, one set of ventilators may be used for several structures or one structure may include large amounts of mechanical equipment requiring large amounts of air. Therefore, the range of sizes are specified as a function of the air volume to be supplied.

6. CONCLUSIONS AND RECOMMENDATIONS

In this chapter the recommendations are summarized for the single personnel shelter to be erected and occupied by an engineer platoon in a forward area. Detailed plans for the proposed shelter are included as Appendix J.

Although it is difficult to estimate precisely the amount of time that might be required for the construction effort, it appears that the structure can be erected by an engineer platoon and occupied within one week's time, without any special equipment.

The structure is designed to be supplied in independent kits, thus permitting the structure to be altered to fit specific requirements. Each kit is complete except for the erection tools, which are provided in a separate kit. The contents of the various kits are as follows:

Kit No.	Contents
1	Erection Kit (Equipment and tools)
2	Main Structure (Ribs and Sills)
3	End of Main Structure and Bulkhead (Ribs, Sills and End Bulkhead)
4	Passageway Structure (Frames, Sheeting and End Bulkhead)
5	Utility Structure (Frames and Sheeting)
6	Entrance Structure (Vertical Pipe and Hatch Cover)
7	Ventilation Structure (Air Ducts)

The basic structure is 16 ft. by 48 ft. in plan and consists of inverted structural tee arches, 8 ft. in radius, spaced 3 ft. on centers. Timber blocks span horizontally between these tees. Although the basic

structure is 48 ft. long, it may be lengthened or shortened by 12 ft. multiples according to the number of kit 2 specified. The foundations consist of two 8 by 12-in. timbers bolted between two 9-in. steel channels, with the channels acting as a splice for the timbers at intervals of 12 ft. This basic structure was selected on the basis of the apparent least weight of the individual parts, the apparent ease of construction, and the high degree of flexibility possible. The structure is designed to resist an overpressure at the surface of 100 psi and the nuclear effects associated with this overpressure, regardless of the size of weapon used in an attack against it. The basic structure and most of its appurtenances can be modified in the field to withstand overpressures and associated effects of as much as 300 psi. (See Appendix H).

The end bulkheads consist of vertical wide-flange posts supporting timber blocks spanning horizontally. These posts are supported by a welded truss spanning between the ends of the sills.

A vertical circular entrance pipe rests upon a horizontal passageway. The vertical entrance pipe actually is composed of two concentric corrugated pipes. The space between the pipes is filled with concrete to prevent buckling of the pipes. Welded rectangular frames support timber sheeting on the sides of the horizontal entrance structure, while metal plates span between the frames to form the roof.

The following specific recommendations are made with regard to further investigations.

(1) A test erection program by an engineer platoon should be initiated to determine not only the actual erection time but the complexity of the erection as well.

(2) A field test program involving actual blast loads should be carried out to validate the structural integrity of the proposed structure. It would be desirable at the same time to test various alternates, such as structures with no bulkheads at all.

APPENDIX A

REFERENCES

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1.2 Merritt, J. L. and Newmark, N. M., "Design of Underground Structures to Resist Nuclear Blast," Vol. II, Final Report, Contract DA-49-129-eng-312, Urbana, 1958, UNCLASSIFIED.

1.3 Newmark, N. M., "Considerations in the Design of Underground Protective Structures," Vol. III, Final Report, Contract AF-04(647)-156 with AMF, 1958, SECRET.

3.1 Department of Defense, "The Effects of Nuclear Weapons," Superintendent of Documents, Washington, 1962, UNCLASSIFIED.

3.2 Department of Defense, "The Capabilities of Atomic Weapons," USGPO, Washington, 1957, CONFIDENTIAL.

3.3 Sinnamon, G. K., Austin, W. J., and Newmark, N. M., "Air Blast Effects on Entrances and Air Intakes of Underground Installations," WT 726, Contract DA-49-129-eng-239, 1955, CONFIDENTIAL, RD.

3.4 Strope, W. E., Porteous, L. G., and Greig, A. L., "Specifications and Costs of a Standardized Series of Fallout Shelters," USNRDL-TR-366, 1959, UNCLASSIFIED.

3.5 Hubbard, Dr. A. W., Private communication to Dr. J. G. Hammer concerning human metabolism.

4.1 American Society of Civil Engineers, "Design of Structures to Resist Nuclear Weapons Effects," Manual of Engineering Practice No. 42, New York, 1961, UNCLASSIFIED.

4.2 American Concrete Institute, "Building Code Requirements for Reinforced Concrete," Journal ACI, Detroit, 1956, UNCLASSIFIED.

APPENDIX B

NOTATION

The following notation is used in this report:

b = width of member

B = span of arch

c = distance from neutral axis to extreme "fiber" in flexural members

c_v = seismic velocity

C = ratio of compression to tension reinforcement in concrete member

d = effective depth of concrete member

D = diameter of entrance structure or ventilator

E = modulus of elasticity

f_a = actual stress induced by axial force on timber or steel

f_b = actual stress induced by flexure on timber or steel

f'_c = standard cylinder strength of concrete

f_v = dynamic yield strength of web reinforcement in concrete

f_y = dynamic yield strength of metal

F = flexural shape factor for metal members

F_a = allowable stress for axial force on timber or steel

F_b = allowable stress for flexure on timber or steel

F_d = drag force acting on ventilator

h_{ave} = average depth of cover over arch

H = height of structure or rise of arch

I = moment of inertia of cross-section

K = lateral force coefficient for peak shock intensity on buried vertical walls

L = length of clear span

L' = total width of structure

m = mass per unit length of element alone

m' = mass per unit length of element and soil overburden

M_d = maximum bending moment produced by drag force on ventilator

M_{max} = maximum bending moment

M_y = the statical moment corresponding to first yield in material

p_a = ambient atmospheric pressure

p_{cr} = critical overpressure determined by buckling

p_d = drag pressure

p_m = peak vertical stress acting on element

p_{so} = peak side-on overpressure at the ground surface

r = yield resistance for an element of any material

r' = radius of arch

r_{dt} = diagonal tension resistance of concrete

r_f = flexural resistance of concrete

r_{sp} = pure shear resistance of concrete

R = radius of gyration

S = maximum thrust in arch

t = thickness of dome

t_d = effective duration of force

t_r = effective rise or application time of force

T = natural period of vibration of element alone

T' = natural period of vibration of element and soil overburden

W = weapon yield

x_m = maximum transient displacement

x_y = yield displacement

λ = parameter defining natural period of vibration in flexure

λ' = parameter defining yield displacement in flexure

μ = ductility ratio = x_m/x_y

σ_y = yield stress of material

φ = percentage of tension reinforcement in concrete

φ_v = percentage of web reinforcement in concrete

APPENDIX C

STRUCTURAL DESIGN CALCULATIONS

The structural design calculations that follow are for the 100 psi overpressure assumed as a static load, in accordance with Section 5.1. However, since the periods of vibrations of the individual members are so short in comparison with the duration of the loading, the static design represents also the final design, in accordance with Sections 5.2 and 5.3.

N. M. NEWMARK URBANA, ILLINOIS	JOB DA-22-079-eng 225 SUBJECT Flexible Underground Structure	SHEET NO. ___ OF 87 BY C.B. DATE 3/9/60 CHKD. BY A.F. DATE ___

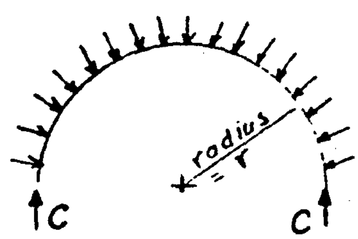
Basic Assumptions

1. Peak overpressure of 100 psi

2. Lateral Dynamic earth pressure equals 50 % of overpressure or 50 psi

3. Dynamic load factor need not be applied except to timber lagging in main arch and end bulkhead, where it equals 1.5

4. That the ring or pipe analogy may be used to compute stress in circular arch



$C = rp\ell$

where:

r = radius of ring

p = radial pressure

ℓ = spacing of ribs

5. The following permissible stresses under dynamic loading for materials indicated.

   a. A-7 Structural Steel
      50,000 psi - bending and direct compression

   b. Southern Pine Timber - Dense structural
      or
      Douglas Fir Timber - B.+S. Dense Construction

      3000 psi - compression ⊥ to grain

      6000 psi - Compression // to grain

      7000 psi - bending

      900 psi - horizontal shear

Basic Assumptions (Cont'd)

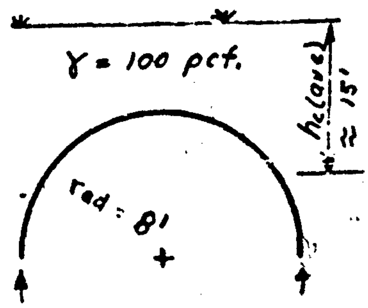
5. Permissible Stresses:

c. Corrugated Plates
Similar to Armco Multi-Plate
or Equal

30,000 psi bending

d. Fillet Welds

29,000 psi net throat area.



Loading

Overpressure 100 psi
Overburden $100 \times \frac{15}{144} \approx$ $\underline{10}$
110 psi

Say ribs @ 36" spacing

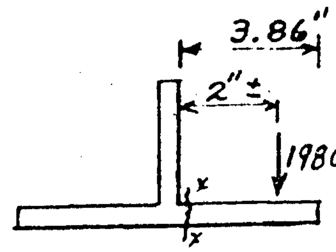
$C = prR$
$= 110 \times 8' \times 12 \times 36 = 380,000$ #

@ 50,000 psi = 7.6 sq ins rib area

@ 3,000 psi = 126 sq ins Bearing PL area

Say ST 4 WF @ 29 #/ft,
Area - 8.53 sq ins
Depth - 4.38"
Flange - 8.22" x 0.808"
Web (t) - 0.510"

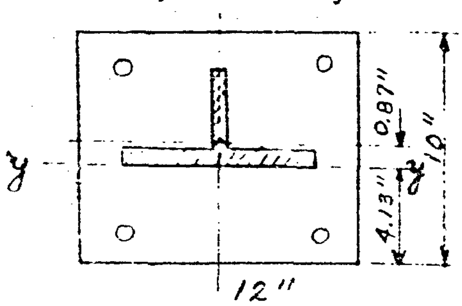
Check moment in outstanding flange leg



$1980^{\#} = 110^{\#} \times \frac{36"}{2}$

$M_x = 1980^{\#} \times 2" \approx 4000$ "#
$\div \frac{1 \times 0.808^2}{6}$ = [illegible] psi
< 50,000 OK

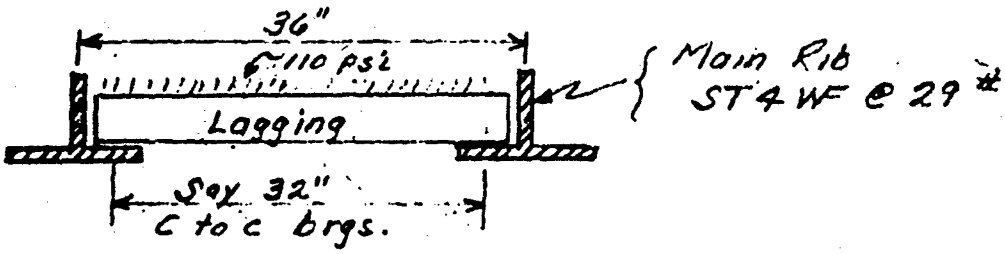
Say Bearing PL 10" x 12" x 1 3/4"



Total Force on PL = 380,000 #
$\div (10 \times 12") = 3180$ psi

Assuming uniform distribution:
M_y = x 12" x 4.13" x $\frac{4.13}{2}$
= 324,000 "#
$\frac{I}{c}$ $\frac{12 \times 1.75^2}{6} = 6.12 \text{ in}^3$ $\div$ 6.12 in^3
= 53,000 psi Say OK

Lagging beween Ribs



Total Forces and Moments (per inch width).

Max. Moment $= \frac{w\ell^2}{8} = 110^{\#} \times \frac{\overline{32}^2}{8} = 14,100$ in·lb

End Reaction $110^{\#} \times \frac{36}{2} = 1980$ lb

Try Corrugated Pls

$14,100^{''\#} \div 30,000$ psi
$= 0.470^{in^3} = \frac{I}{c}$

Per page 78 of 1958 Armco "Handbook of Drainage and Construction Products" it would require 4 "MULTI-PLATE" sections of No 5 gage stacked to resist this loading.

Thickness = 0.215 in per Pl
Proj. Area = 0.267 sq in/in "

Plates Weight = 0.267 × 4 × 40.8 #/sq ft = 43.7 lb/sq ft covered.

Try Timber Lagging

$14,100^{''\#} \times 1.5$ (Dyn. Ld. Fact.)
$= 21,200^{''\#}$
@ 6000 psi
$= 3.52^{''^3} = \frac{I}{c} = S$

Note: 6000 psi used so as to allow for axial stress in blocks

$\overline{thick}^2 = \frac{6S}{width} = \frac{6 \times 3.52}{1''} = 21.2\ in^2$

$\therefore$ thickness $= \underline{\underline{4.6''}}$

Check timber bearing (Crushing)
End reaction = 1980 lbs
÷ say 3¾ in = 530 psi

OK for both ∥ and ⊥ grain

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URBANA, ILLINOIS

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SUBJECT Flexible Underground Shelter
Main Arch

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BY JnB DATE ____
CHKD. BY ____ DATE ____

Timber Lagging (Cont'd)

Check Horizontal Shear

Assume rectangular section 1" x 4.6"
Actual timber to be tapered or beveled.

Horizontal shearing stress =

$$v = \frac{VQ}{Ib} = \frac{3}{2} \times \frac{V}{bd} \text{ (for rectangular sections)}$$

$$= 1.5 \times \frac{1980^{\#}}{1'' \times 4.6''} = 640 \text{ psi static load}$$

If dynamic load factor is considered } $\times 1.5 = 960$ psi
> 900 psi allow.

Use 6" nominal depth for timber lagging
Stress grade (5½" actual)

Loads

Per foot of sill = $110^{lb/sq\,in} \times 96'' \times 12 = 127^{K}/ft$

Width

2 ft wide sill gross brg press = $\div 2 \approx 64^{K}/sq\,ft$

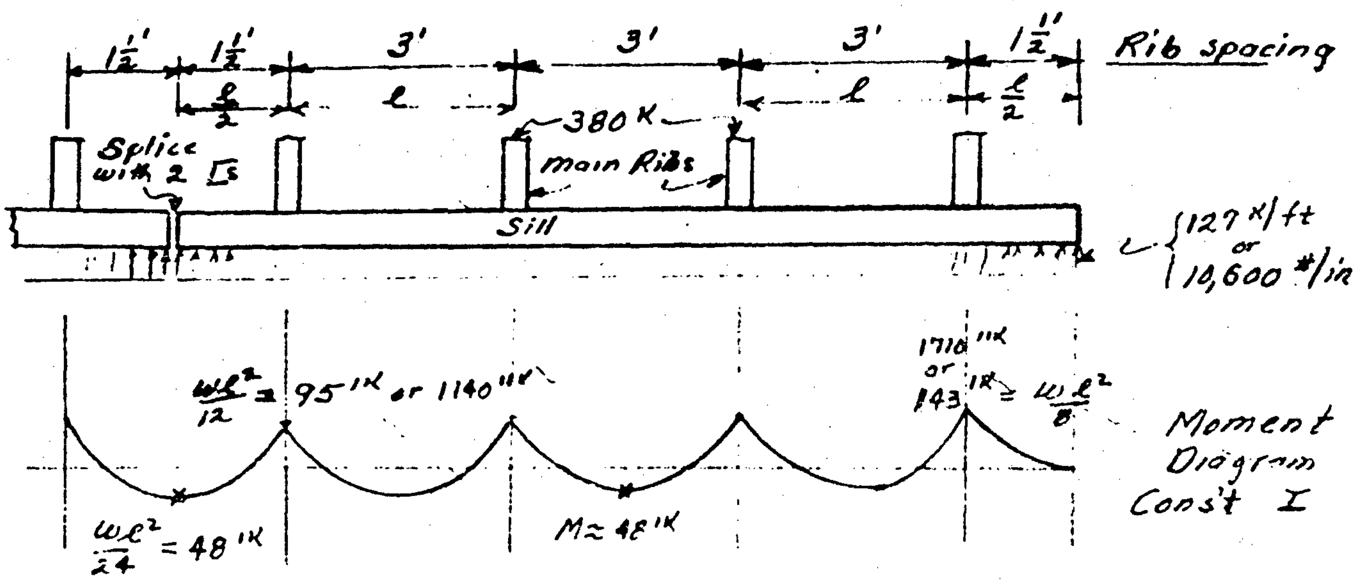
overall blast pressure =

$100\ psi \times 144\ sq\,in \approx -14^{K}/sq\,ft$

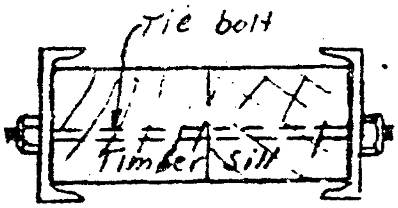
Net Bearing Pressure = $50^{K}/sq\,ft$

Depth

Assume 2 [s to splice ends of 12' sills



Note:
Assume constant I through out.
[s to splice sills at end of 12' sills
Sills to splice [s at end of 12' [s



Sill-End View

Max. Moment in steel [s
$= 48^{'K}$ or $580^{''K}$
@ 50,000 psi = $\frac{I}{c} = 11.6\ in^3$ (2 [s)

Max. Moment in Sill

Total	=	$143^{'K}$ or $1,720^{''K}$
in [s	=	48 or 580
In Sill	≈	$95^{'K}$ or $1,140^{''K}$

@ $1880 = 605\ in^3 = \frac{I}{c} = S$

$d^2 = \frac{6S}{b} = \frac{6 \times 605}{24} = 151\ in^2$

$d \approx 12.5''$

This is min. d if min [s used.

Timber stress:

$= 50,000 \times \frac{E_t}{E_s} \times (say\ \frac{3}{4})$

$= 50,000 \times \frac{1,500}{30,000} \times \frac{3}{4} = 1880\ psi$

where: $\frac{3}{4}$ = estimate depth of timber compared with [s

Depth (Cont'd)

Try 2 [s 10 x 2 5/8" @ 15.3 # $\frac{I}{c} = 2 \times 13.4 = 26.8\ in^3$
9" x 24" timber @ 60 #
(8 1/2 x 23 1/2") $\frac{I}{c} = \frac{23.5 \times 8.5^2}{6} = 283.0\ in^3$

Resisting Moment
2 [s 10 x 2 5/8 $26.8 \times 50{,}000\ psi = 1{,}340{,}000''\#$
9 x 24 timber $283. \times 50{,}000 \times \frac{4500}{30{,}000} \times \frac{4.25}{5.00} = \underline{600{,}000}$
$1{,}940''K$

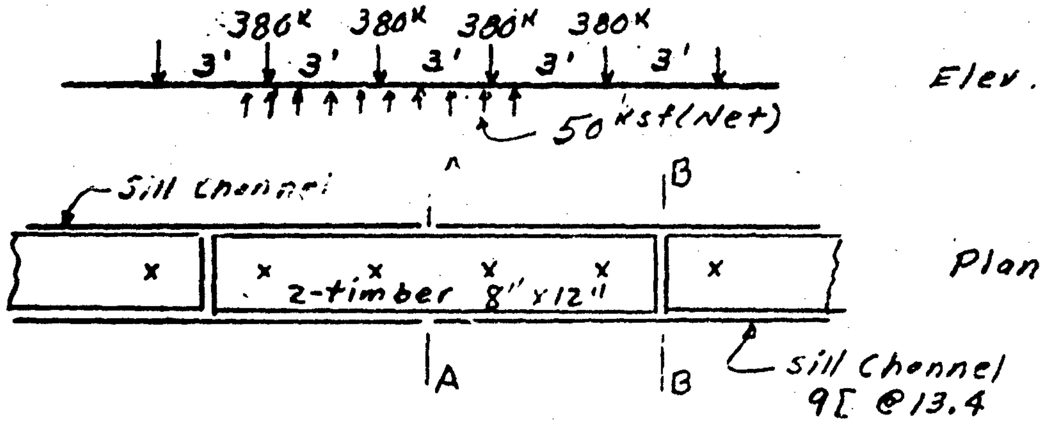
This is the section required to resist the moment in the sill under the first main arch rib. The sill moment is a maximum at this point. It decreases to approximately 1140 ''K at the interior ribs. This is based on the assumption that there is constant I throughout. Since neither the timber sill nor the [s are continuous the actual moment at the discontinuities will tend to be reduced and the moment under the arch ribs increased slightly. There is no reason to believe that the full moment of 1710 ''K will be developed under the first arch rib. Thus, it would appear that the overall section could be designed for less than this amount. Sufficient section of both [s and sill alone must be provided to resist a moment of 48 K or 570 ''K

Try 2 [s 9 x 2 1/2" @ 13.4 # $\frac{I}{c} = 2 \times 10.5 = 21.0\ in^3$
and 2 - 8" x 12" timbers @ 50 #(±) $\frac{I}{c} = 2(11.5 \times \frac{7.5^2}{6}) = 216\ in^3$
7 1/2 x 11 1/2

Resisting Moment
2 [s 9 x 2 1/2 $21.0\ in^3 \times 50{,}000 = 1{,}050''K$
2 8x12 timbers $216 \times 50{,}000 \times \frac{1500}{30{,}000} \times \frac{3.75}{4.50} =$
or (2080 psi) $= \underline{450}$
$1{,}500''K$

Stresses at discontinuities
Moment = 48 'K or 570 ''K
$\div 21.0\ in^3 = 27{,}000\ psi$ in [s OK
$\div 216\ in^3 = 2630\ psi$ in Sill OK

Check Horizontal shear in Sill



As vertical shear at sections A-A and B-B is ≈ zero (by symmetry), these sections at the splices could not be critical. Horizontal shear could only be critical at the loading points under the arch ribs, where both timbers and channels are available for resistance

Check two conditions:
1) Neglect [s - Timbers only

2) Consider both timber and [s.

1) Timber sills only - $7\frac{1}{2}$" x 23" Net

$$H = \frac{VQ}{It} \quad \text{reduces to} \quad \frac{3V}{2bd} \quad \text{for rect. beams}$$

$$\therefore H = \frac{3 \times \frac{380}{2}}{2 \times 7.5'' \times 23''} = 1.65 \text{ Ksi}$$

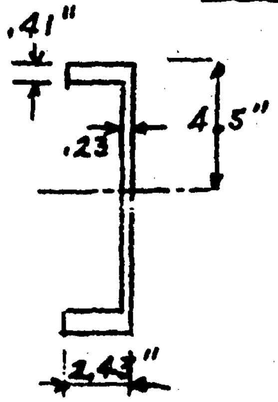
Probably high if timber only.

2) Consider both timber and [s
Assume $E_t \approx 1,500,000$ psi
$\therefore n \approx 20$

Total transformed width =
23" + 2 x 20 x 0.23" = 32"
(timber) (web)

Horizontal Shear Cont'd.:

Q value



[s. $0.41'' \times 2.2'' \times 4.3'' = 3.9$

$0.23 \times \frac{(4.5)^2}{2} = \underline{2.3}$

6.2 $''^3$/[each

Transformed = $\times 2 \times 20 = 250$

Timber = $\frac{\overline{3.75}^2}{2} \times 23 = \underline{160}$

$410''^3$

I value

[s $I = 47.3''^4$

Transformed = $\times 2 \times 20 = 1880''^4$

Timber

$\frac{(7.5)^3 \times 23}{12} = \underline{810}$

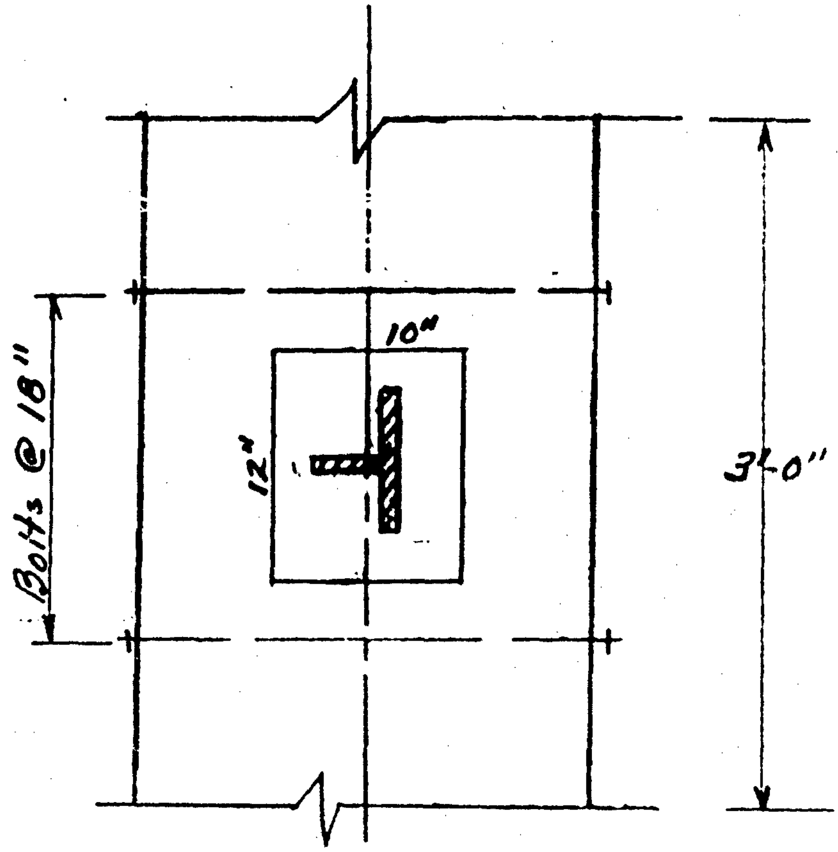
$2690''^4$

$$H = \frac{VQ}{It} = \frac{\frac{380}{2} \times 410}{2690 \times 32} = \underline{\underline{0.9}} \text{ ksi}$$

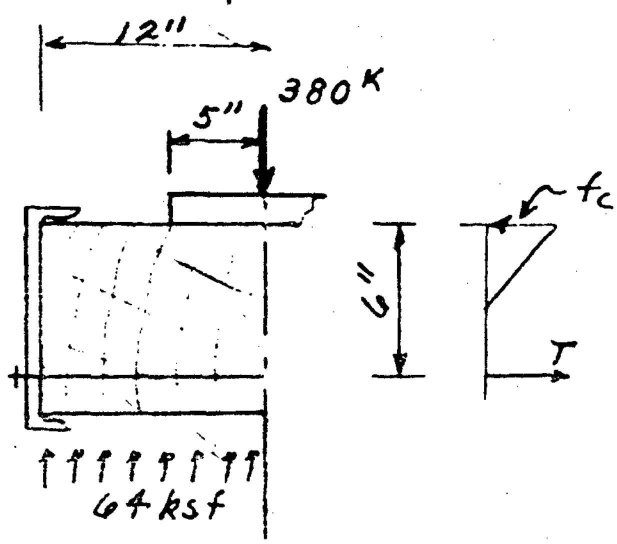
OK for shear

Use 2-9" [s @ 13.4'
2 - 8" x 12" timbers

Check lateral crushing in sill and sill bolt stress



Plan View of Sill



Half end view

℄ Moment:

$$\frac{380}{2} \times 6'' = 1140''^{K}$$

$$-\frac{380}{2} \times 2.5'' = \underline{480}$$

$$\underline{660}''^{K} \quad \text{critical}$$

Mom @ Edge of ℞

$$\left(64 \times \frac{7}{12} \times 3\right) \times 3.5'' = 390''^{K}$$

Compute bolt stress and timber crushing

Assume linear stress-strain relationship to yield. Use transformed Area method

Try 1"ϕ bolts at 18" (see sketch on preceding page)

per area = 0.78 sq "

Steel ratio $\rho = \frac{0.78}{6'' \times 18''} = 0.0072$

If modular ratio $n \approx 20$
$pn = 0.14$

$\therefore k = \sqrt{\overline{pn}^2 + 2pn} - pn = \sqrt{(.14)^2 + .28} - .14 = 0.41$

$T = C = \left(\frac{f_c}{2} kd\right) b = \frac{f_c}{2} \times .41 \times 6'' \times 18'' = 22 f_c$ per bolt

Resisting Moment

$= M_r = 22 f_c \times 6\left(1 - \frac{.41}{3}\right) = 113 f_c$ per bolt   (.86)

also $= \frac{660}{2}$ ''K

$\therefore f_c = \frac{660''K}{2 \times 113} = \underline{\underline{2.92}}$ ksi (timber)

OK as well below 7.0 ksi
$\therefore$ Use of n is reasonable

$T = 22 f_c = 2.92 \times 22 = 64^K$/bolt

$\therefore f_s = \frac{64^K}{.78} = 83$ ksi (steel - nominal diameter)

Effective area = 0.606 sq ins (ASTM-325)

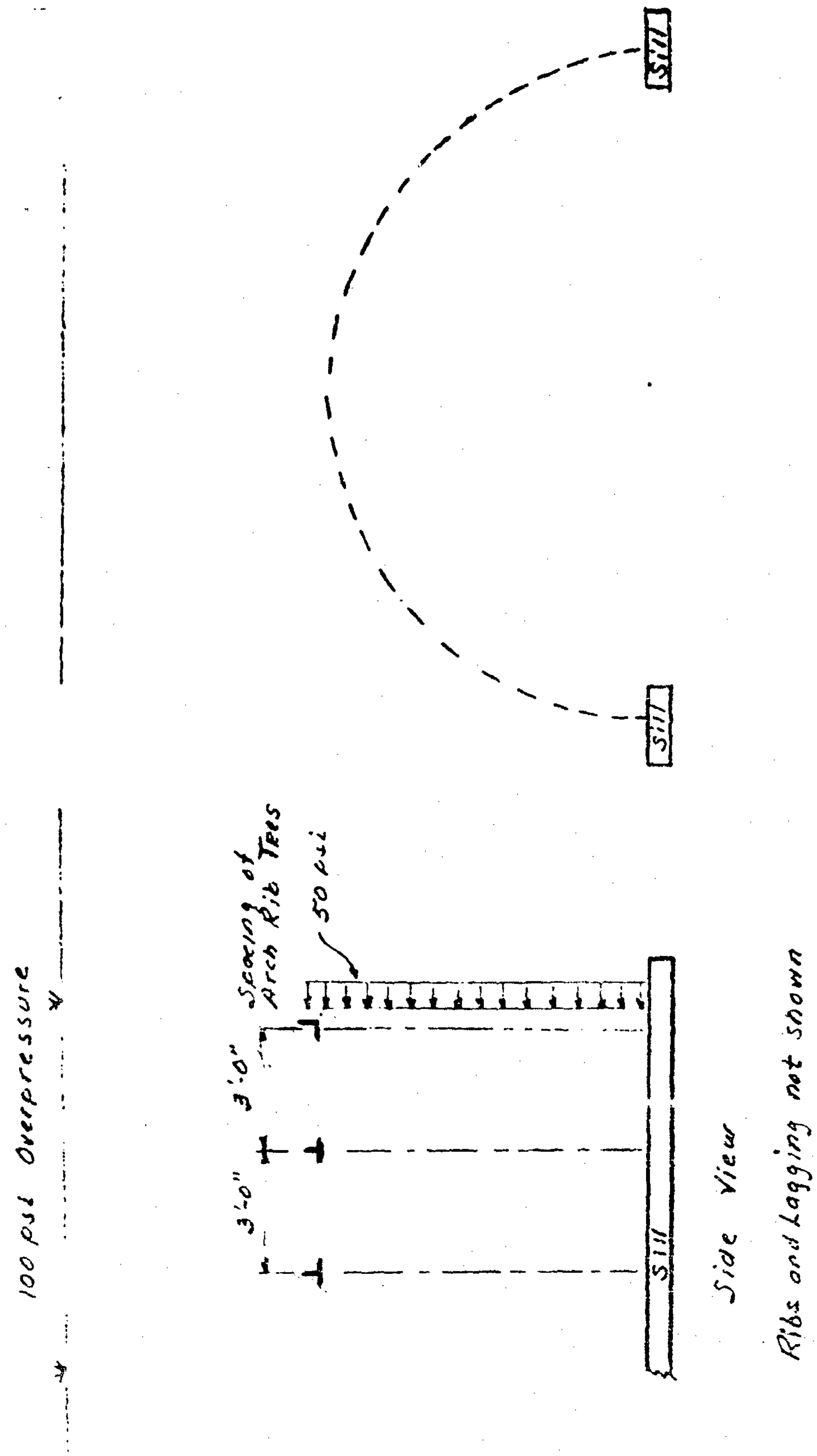
$\therefore f_s$ (effective) $= \frac{64}{.606} = 105$ ksi

Say OK - ASTM 325 - gives static tensile strength of 1"ϕ = 69.7^K

N. M. NEWMARK
URBANA, ILLINOIS

JOB DA-22-079-eng 225
SUBJECT Flexible Underground Structure
End Bulkhead

SHEET NO. ____ OF 98
BY J. W. B. DATE ____
CHKD. BY AFD DATE ____

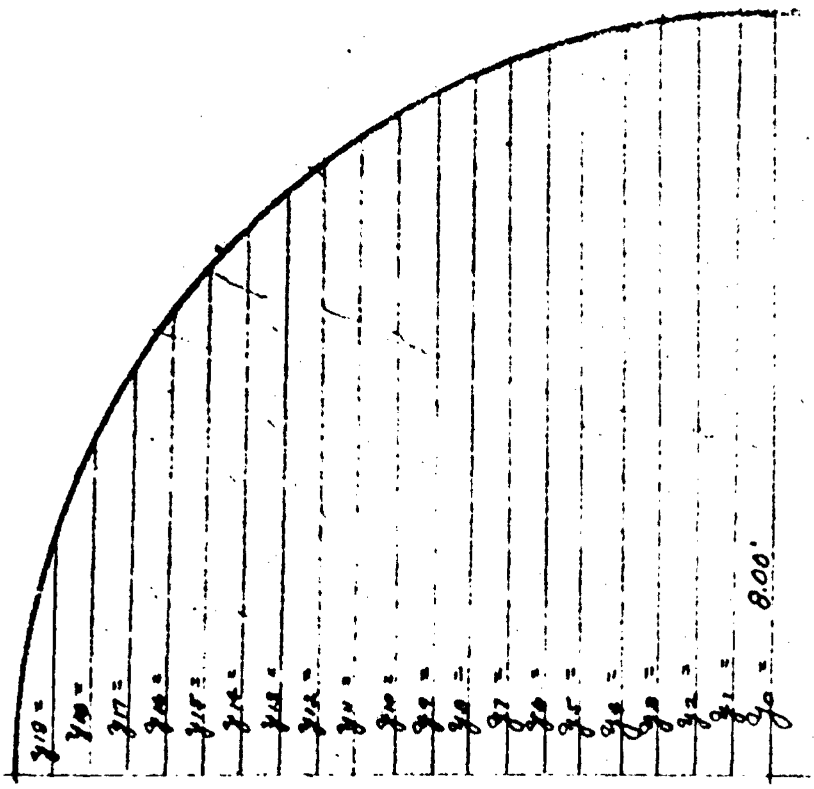


N. M. NEWMARK
URBANA, ILLINOIS

JOB DA-22-079-eng 225
SUBJECT Flexible Underground Structure
End Bulkhead

SHEET NO. 2 OF 98
BY JWB DATE
CHKD. BY AFP DATE

Arch Rib Ordinates



20 @ 0.4' = 8.0'

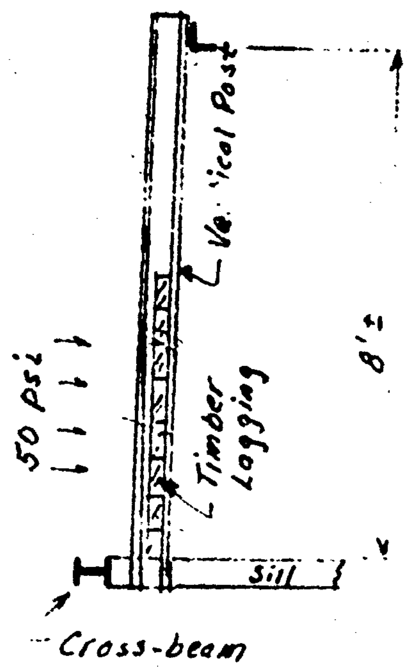
$y_0 =$			$8.00'$	$y_1 = \sqrt{8.0^2 - 0.4^2}$	$= \sqrt{63.84}$		$= 7.99'$
$y_2 = \sqrt{8.0^2 - .8^2}$	$= \sqrt{63.36}$		$= 7.96$	$y_3 =$	-1.2^2	$= 62.56$	$= 7.91'$
$y_4 = \sqrt{8.0^2 - 1.6^2}$	$= \sqrt{61.44}$		$= 7.84$	$y_5 =$	-2.0^2	$= 60.00$	$= 7.74$
y_6	-2.4^2	$= 58.24$	$= 7.63'$	$y_7 =$	-2.8^2	$= 56.16$	$= 7.49$
y_8	-3.2^2	$= 53.76$	$= 7.33$	$y_9 =$	-3.6^2	$= 51.04$	$= 7.14$
y_{10}	-4.0^2	$= 48.00$	$= 6.93$	$y_{11} =$	-4.4^2	$= 44.64$	$= 6.68$
y_{12}	-4.8^2	$= 40.96$	$= 6.40$	$y_{13} =$	-5.2^2	$= 36.96$	$= 6.08'$
y_{14}	5.6^2	$= 32.64$	$= 5.71$	$y_{15} =$	-6.0^2	$= 28.00$	$= 5.29'$
y_{16}	6.4^2	$= 23.04$	$= 4.80$	$y_{17} =$	-6.8^2	$= 17.76$	$= 4.21$
y_{18}	7.2^2	$= 12.16$	$= 3.49$	$y_{19} =$	-7.6^2	$= 6.24$	$= 2.50'$

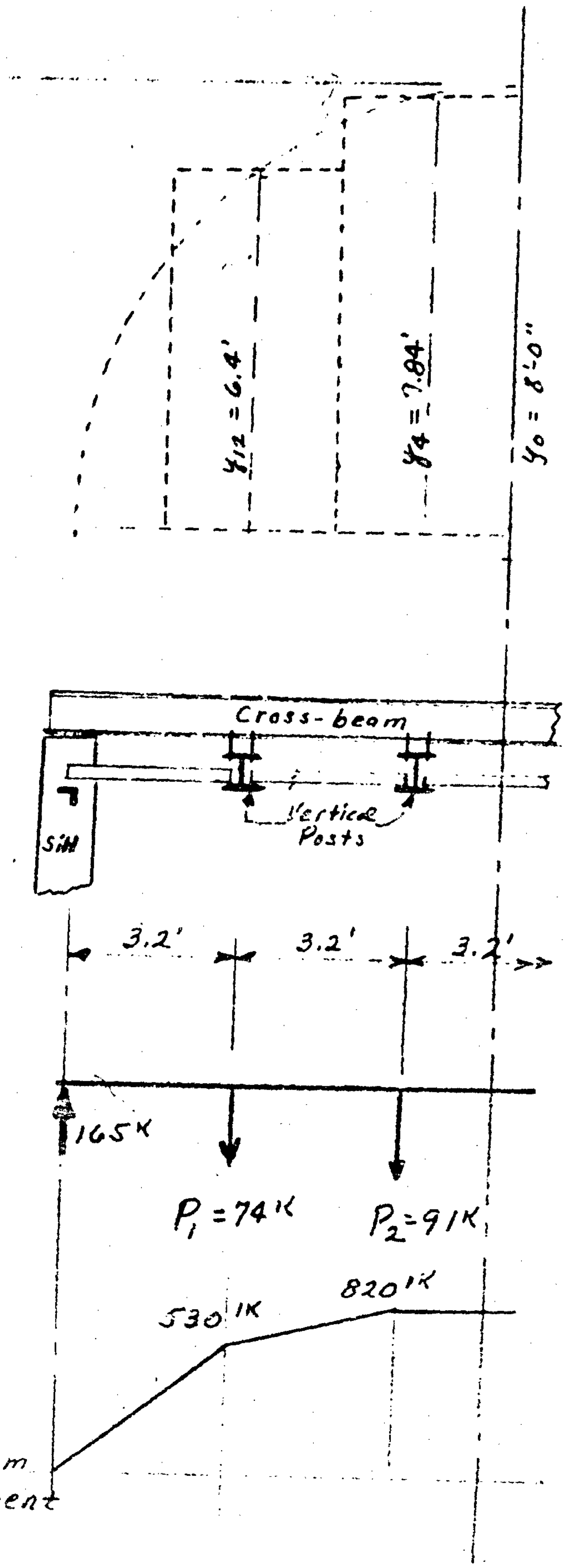
N. M. NEWMARK
URBANA, ILLINOIS

JOB: DA-22-079-eng 225
SUBJECT: Flexible Underground Structure
End Bulkhead

SHEET NO. ____ OF 100
BY: JMB DATE ____
CHKD. BY: AFP DATE ____

Cross-beam





cross-beam moment

Post Loads

$50 \text{ psi} \times (3.2 \times 12)$
$= 1.92^{K/''}$ at post

$\times \frac{7.74 \times 12}{2} \approx 91^K = P_2$

$\times \frac{6.4 \times 12}{2} \approx 74^K = P_2$

N. M. NEWMARK
URBANA, ILLINOIS

JOB DA-22-079-eng 225 SHEET NO. ____ OF 101
SUBJECT Flexible Underground Structure BY JMB DATE 4/19/62
End Bulkhead CHKD. BY AEO DATE ____

Timber Lagging

Check timber depth for 3.2' spacing

$$50 \text{ psi} \times \frac{37^2}{8} = 8{,}600 \text{ ''lb/in width}$$

$\times 1.5$ (Dynamic Load factor)

$= 12{,}800$ ''#

@ 7000 psi $= 1.85 \text{ in}^3 = \frac{I}{c}$

$$d^2 = 6\,\frac{I/c}{b} = \frac{6 \times 1.85}{1''} = 11.1 \text{ in}^2$$

$\therefore d = 3.33''$

Could use 4" nominal timber - stress grade

Check timber depth if post spacing = 4.0'

$$50 \text{ psi} \times \frac{48^2}{8} = 14{,}400 \text{ ''lb/inch width}$$

$\times 1.5$ (Dynamic Load factor)

$= 21{,}600$ ''lb

@ 7000 psi $= 3.08 \text{ in}^3 = \frac{I}{c}$

$$d^2 = 6\,\frac{I/c}{b} = \frac{6 \times 3.08}{1''} = 18.5 \text{ in}^2$$

$\therefore d = 4.3''$

Vertical Posts

If spaced at 3.2'

$1.92\ ^{k}/in$ of post

$\times$ say $\frac{\overline{100}^2}{8} = 2400\ ^{''k}$

@ 50 ksi

$= 48\ in^3 = \frac{I}{c}$

Say 16 WF @ 36#

$I/c = 56.3\ in^3$

If post spaced at 4.0'

$50\ psi \times (4 \times 12) = 2.4\ ^{k}/in$

$\times$ Say $\frac{\overline{100}^2}{8} = 3000\ ^{''k}$

@ 50 ksi =

$60\ in^3 = \frac{I}{c}$

Say 16 WF @ 40#

$I/c = 64.4\ in^3$

flange = $\frac{1}{2}'' \times 7''$

Web = $5/16''$

Cross-beam

If Posts spaced at 3.2'

Max Mom = 820 'k or 9850 ''k

@ 50 ksi

$= 197\ in^3$

24 WF @ 84#

$I/c = 196.3\ in^3$

Total beam weight = $84 \times 18' \pm = 1500$#

Note: While this is a heavy beam,
it could be handled by no more than 10 men.

Try other configurations

N. M. NEWMARK	JOB DA-22-079-eng 225	SHEET NO. ___ OF 103
URBANA, ILLINOIS	SUBJECT Flexible Underground Structure	BY JWB DATE 5/8/62
	End Bulkhead	CHKD. BY AFP DATE

Cross-beam (Cont'd)

If post spaced at 4' as shown.

P_1

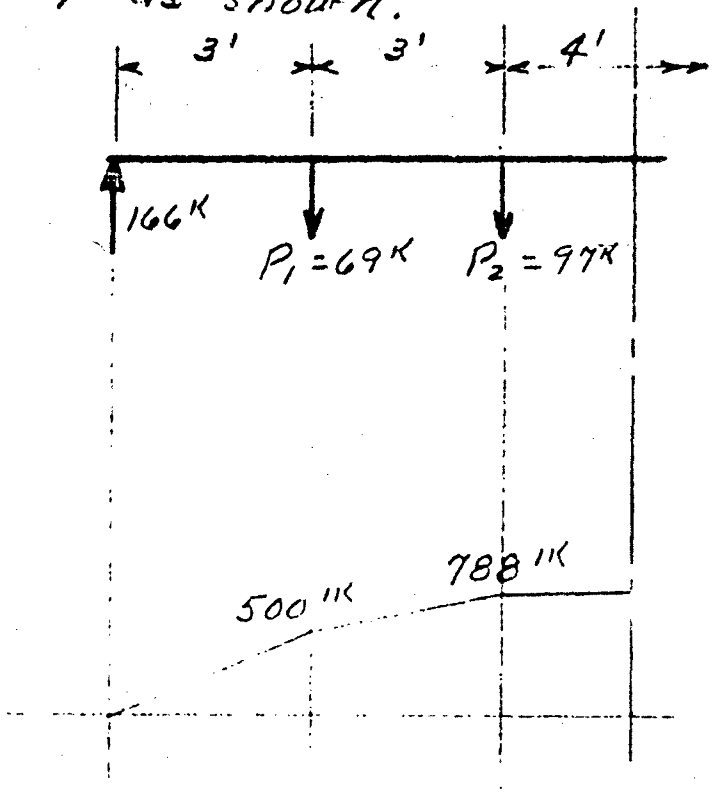
$50\,psi \times (3' \times 12) \times \overset{(9.12\pm)}{\frac{6.4 \times 12}{2}}$

$= 67^K - P_1$

P_2

$50\,psi \times (3.5 \times 12) \times \overset{(45)}{\frac{7.74 \times 12}{2}}$

$= 97^K = P_1$



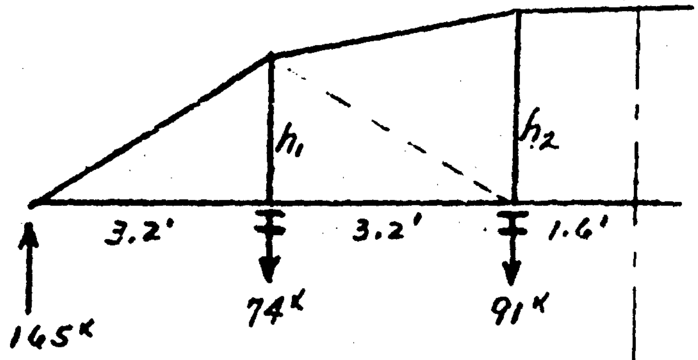
Max. Moment = $788'^K$ or $9500''^K$

@ 50 ksi

$= 190\ in^3 = I/c$

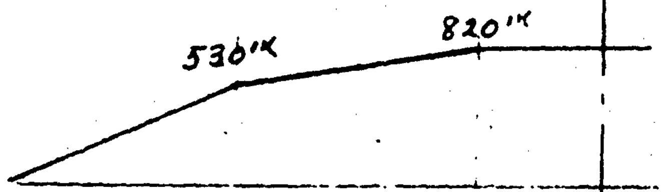
Try truss instead of Cross-beam

If posts spaced at 3.2'

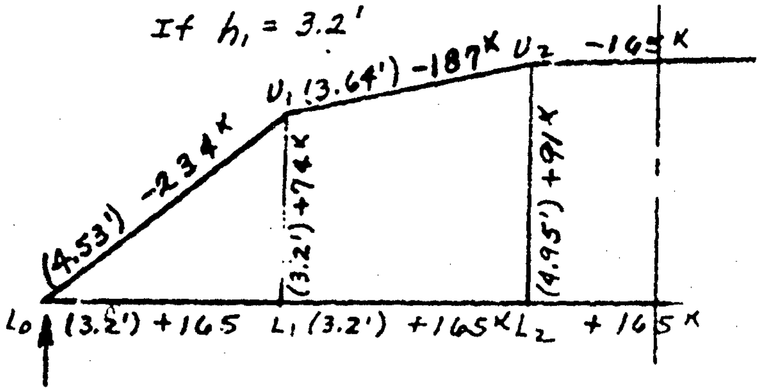


Make h_1 and h_2 such that diagonal has no stress.

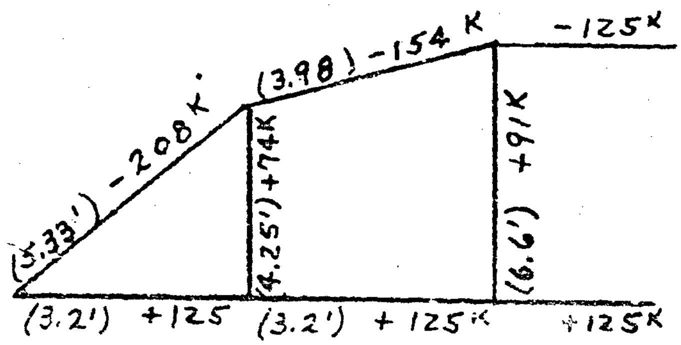
If $h_1 = 1$
then
$$h_2 = \frac{820}{530} = 1.55$$



If $h_1 = 3.2'$

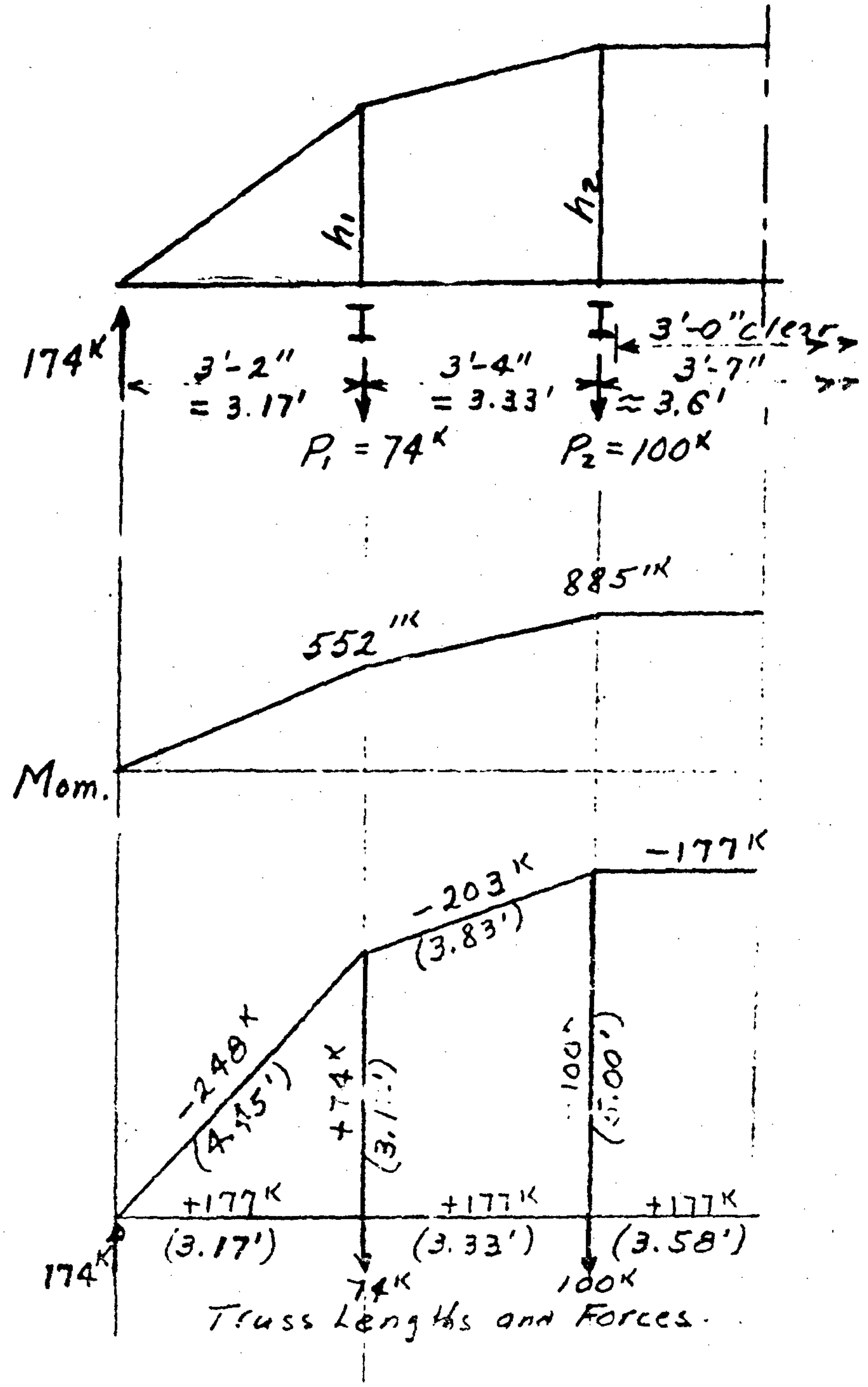


If $h_1 = 4.25'$

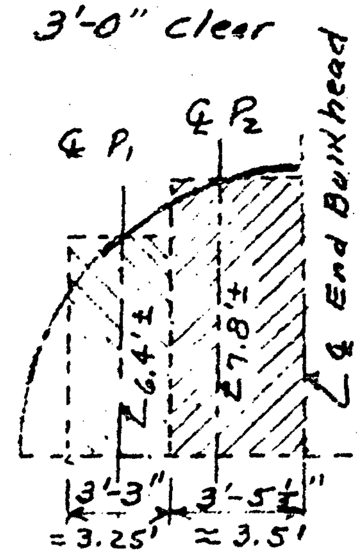


End Truss (Cont'd)

Adjust center panel to permit 3'-0" clear between posts for entrance way.
Say 16'-7" c to c of sills



Truss Lengths and Forces.



Areas Contributing to P_1 + P_2

$P_1 = 50\,psi \times 3.25' \times 12 \times \dfrac{6.4'\pm \times 12}{2} \approx 74^K$

$P_2 = 50\,psi \times 3.5 \times 12 \times \dfrac{7.8'\pm \times 12}{2} \approx 100^K$

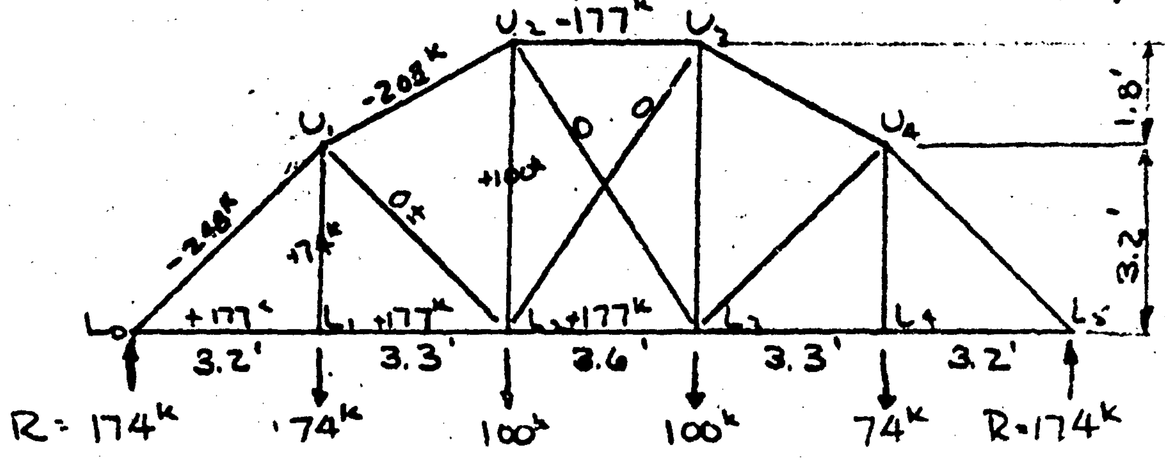
Select h_1 and h_2 such that diagonal has no stress

$\therefore h_2 = \dfrac{885}{552} h_1 \approx 1.6 h_1$

If $h_2 = 5.0'$

$h_1 = \dfrac{5.0}{1.6} \simeq \underline{\underline{3.12}}$

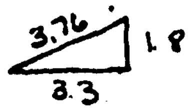
Note:
h_1 subsequently changed to 3.2'± or 3'-2½".



$\underline{U_2U_3}$

$$\frac{174 \times 6.5 - 74 \times 3.3}{5.0} = \frac{1130 - 244}{5.0} = 177$$

$\underline{U_1U_2}$



$$177 \times \frac{3.76}{3.3}$$

DESIGN

L_0L_1, L_1L_2, & L_2L_3 Load = 177^k

Req'd Area = $\frac{177}{50}$ = $3.54^{\square''}$, Use 2 ⅃L's $5 \times 3 \times \frac{5}{16}$ (A = $4.8^{\square''}$)

L_0U_1 Load = -248^k

Req'd Area = $\frac{248}{50}$ = $4.96^{\square''}$, Use 2 ⅃L's $5 \times 3 \times \frac{5}{16}$ (Stress = 51^{ksi})

U_1U_2, U_2U_3 same as L_0U_1

U_2L_2 Load = 100^k

Req'd Area = $\frac{100}{50}$ = $2^{\square''}$. Use 6 [8.2, (Area = $2.39^{\square''}$)

U_1L_1 same as U_2L_2

U_1L_2 use 2 - 6 [8.2 although stress is insignificant. Possible unsymmetrical loading may cause more stress in U_1L_2

U_2L_3 although there is no stress for the case of a symmetrical loading provision is made for the case of shear in the panel by adding bars. Use Bar $\frac{1}{2} \times 1\frac{1}{2}$

WELDED CONNECTIONS

Allowable stress on Fillet Weld = 29^{ksi} at Throat

$\frac{5}{16}$ Weld has allowable load of $9^{k}/in$

Top and bottom chords are continuous members and if cut are full butt welded.

Web members shall have sufficient $\frac{5}{16}$ weld to develop stress shown on truss.

BOLTED CONNECTION OF 16WF TO TRUSS

BEAM REACTION = 100^{k}

Approx. Ult Shear Strength of $\frac{7}{8}$ Bolt = 40^{k}
Use 25^{k}/bolt to determine number req'd

Bolts Req'd $\frac{100}{25} = 4 - \frac{7}{8}{}''\phi$

ENTRANCEWAY

THE MINIMUM INNER DIAMETER OF THE VERTICAL ENTRANCE PIPE IS SELECTED AS 32" FOR EASE OF ACCESS.

DESIGN OF VERTICAL ENTRANCE PIPE - MULTIPLATE

BELOW A DEPTH EQUAL TO THE DIAMETER OF THE PIPE, I.E., 32", THE TUBE IS SUBJECTED TO A UNIFORM RADIAL PRESSURE OF ONE-HALF OF THE PEAK PRESSURE, I.E., 50 PSI.

THUS, REQUIRED CROSS-SECTIONAL AREA.

$$A_r = \frac{pr}{\sigma} = \frac{50 \times 16}{30,000} = 0.027 \text{ IN}^2/\text{IN}$$

ABOVE A DEPTH EQUAL TO THE DIAMETER, THE TUBE IS ASSUMED TO BE SUBJECTED TO A UNIFORM RADIAL PRESSURE OF 50 PSI, AND A SINUSOIDALLY VARYING RADIAL PRESSURE OF 50 PSI

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1 \quad \text{(VERY NEARLY)}$$

$$f_a = \frac{pr}{A} = pr/t \quad /\text{IN}$$

$$f_b = \frac{Mc}{I}, \text{ WHERE } M = \tfrac{1}{3} pr^2 \quad \text{(EQN 4.15 P 58)}$$

$$I = K^2 A \approx 0.7^2 A \approx 0.5\,t \;/\text{IN}$$

$$c = 1''$$

$$\therefore f_b = \frac{2}{3} \frac{pr^2}{t} \quad /\text{IN}$$

FOR $F_a = F_b = 30,000$ PSI, REQUIRED $t = 0.311$ IN

OR $A = 0.311 \text{ IN}^2/\text{IN}$

∴ REQUIRE 1 GA STD. ARMCO MULTIPLATE OR SIMILAR MATERIAL FOR VERTICAL TUBE $(A = 0.343 \text{ IN}^2/\text{IN})$

DISCUSSIONS WITH ARMCO REPRESENTATIVE INDICATE IMPOSSIBLE TO FORM 1 GA MULTIPLATE INTO TUBE OF 32" DIAMETER AND RETAIN CORRUGATIONS. TO FABRICATE SPECIAL MANDRELS WOULD BE EXCESSIVELY COSTLY UNLESS WERE TO PRODUCE MANY UNITS.

DESIGN OF VERTICAL ENTRANCE PIPE – STEEL CYLINDER

$$f_a = pr/t$$

$$f_b = \frac{Mc}{I} \qquad \text{WHERE } M = \tfrac{1}{3} pr^2$$

$$I/c = S = t^2/6$$

$$\therefore \quad pr/t + \frac{1/3\, pr^2}{t^2/6} = 50{,}000 \text{ PSI (ASTM-A7)}$$

$$\frac{50 \times 16}{t} + \frac{2 \times 50 \times 16^2}{t^2} = 50{,}000 \;; \therefore t = 0.560 \text{ IN}$$

USE 5/8 IN SHEET

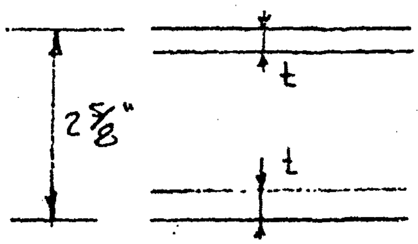
$$\text{WEIGHT} \cong \pi d t L \times 490 \text{ PCF}$$

$$\cong \pi \times \frac{33}{12} \times \frac{5}{96} \times 3 \times 490 \cong 600 \text{ lb.}$$

INADVISABLE AS NOT ONLY HEAVY & DIFFICULT TO HANDLE, BUT ALSO ABOUT 3% STRAIN IN FORMING – INVESTIGATE FURTHER.

DESIGN OF VERTICAL ENTRANCE PIPE – DOUBLE CYLINDER

INVESTIGATE DOUBLE WALLED SYSTEM. ASSUME MEMBERS SPACED 2 5/8" APART. MEAN DIAMETER INCREASED TO APPROXIMATELY 36".



$$f_a = \frac{P}{A} + \frac{Mc}{I}$$

$$30{,}000 = \frac{50 \times 18}{2t} + \frac{1/3 \times 50 \times 18^2}{2t(1\tfrac{5}{16} - t)^2} \times 1\tfrac{5}{16}$$

ASSUMING $t/2 = 0$, APPROXIMATE $t = 0.0835$ IN

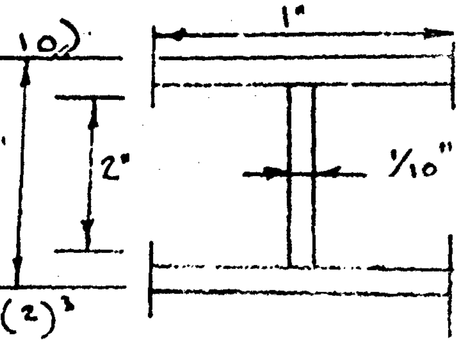
TRY 14 GA, $t = 0.0747$ IN

$$f = \frac{50 \times 18}{2 \times 0.0747} + \frac{1/3 \times 50 \times 18^2 \times 1\tfrac{5}{16}}{2 \times 0.0747 \times 1.2752^2}$$

$$= 6.03 + 29.40 = 35.43 \text{ KSI}$$

ADDING CONCRETE BETWEEN 2-14 GA. TUBES AND ASSUMING ONLY 2" OF 2 5/8" EFFECTIVE STRUCTURALLY:

TRANSFORMED SECTION (n = 10)



$$A_T = 2(0.0747) + 2(.01)$$
$$= 0.15 + 0.2$$
$$= 0.35 \ \text{IN}^2$$

$$I_T = 2(0.0747)(1.2752)^2 + \tfrac{1}{12}(0.1)(2)^3$$
$$= 0.242 + 0.067$$
$$= 0.309 \ \text{IN}^4$$

$$\therefore f = 6.03 \times \frac{0.15}{0.35} + 29.40 \times \frac{0.242}{0.309}$$

$$= 2.6 + 23.0 = 25.6 \text{ KSI} < 30 \text{ KSI}$$

∴ USE 2-14 GA. SECTIONS FILLING ANNULUS WITH CONCRETE

MEAN DIA OUTTER	=	3'- 2 5/8"	SELECTED TO FIT ON COVER PLATE ON ACCESS SECTION
MEAN DIA INNER	=	2'- 9 3/8"	

DESIGN OF HATCH FOR ENTRANCEWAY

IN ORDER TO NOT HAVE A VERTICAL LOAD SUPERIMPOSED ON THE ENTRANCE PIPE, THE HATCH AND ITS SUPPORTING FRAMEWORK ARE SEPARATED FROM THE ENTRANCE PIPE.

THE HATCH DIAMETER IS SELECTED AS 3'-0" IN ORDER TO PROVIDE SUFFICIENT BEARING AREA & STILL HAVE ABOUT 2'-6" CLEAR FOR ACCESS. WITH THIS AS THE MINOR DIAMETER, A MAJOR DIAMETER OF 60" RESULTS IN A CROWN RISE OF 7" WHICH IS CONSIDERED SATISFACTORY

REQUIRED THICKNESS OF DOMED COVER FOR SIDE-ON OVERPRESSURE OF 100 PSI AND COVER FABRICATED OF A7 STEEL, OR EQUIVALENT, IS

$$t = \frac{pr}{2f} = \frac{100 \times 30}{2 \times 50{,}000} = 0.030''$$

WHICH IS MUCH TOO THIN TO WELD TO FOR FASTENING HINGES AND FOR POSSIBLE LOCALIZED LOADING. THEREFORE, USE A THICKNESS OF $t = 1/4''$.

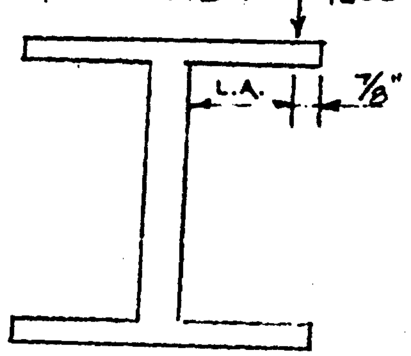
IN ORDER TO STIFFEN THE HATCH, A $1\frac{3}{4} \times 1\frac{1}{4} \times \frac{1}{4}$ CIRCUMFERENTIAL ANGLE IS ARBITRARILY SELECTED AND WELDED CONTINUOUSLY TO THE HATCH.

HATCH SUPPORT FRAME

INVESTIGATE WF SECTION AS SILL TO SUPPORT HATCH. TREATING SILL FLANGE AS A CANTILEVER, THE TOTAL LOAD IS $\pi r^2 p$ AND THE LOADED CIRCUMFERENCE IS $2\pi r$. ALLOWING 6" ON EACH SIDE FOR INSTALLATION OF LATCH ASSEMBLIES, DIAMETER OF SILL WEB = 3'0 + 2×6" = 4'-0"
THEREFORE, LOAD/INCH IS

$$\frac{pr}{2} = 100 \times \frac{4 \times 12}{4} = 1200\ \#/\text{IN}$$

IF TREAT AS CONCENTRATED LOAD ½ (1¾") OR 7/8" FROM FLANGE EDGE: 1200 #/IN



$$L.A. = \frac{\text{FLANGE WIDTH} - \text{WEB}}{2} - \tfrac{7}{8}"$$

TRY 8 WF 58

$$L.A. = \frac{8.222 - 0.510}{2} - \tfrac{7}{8} = 3.08"$$

$$M = 1200 \times 3.08 = 3{,}700$$

$$S = \frac{bt^2}{6} = \frac{1 \times 0.808^2}{6} = 0.108$$

$$f = \frac{M}{S} = \frac{3{,}700}{0.108} = 34{,}200 \text{ psi ok}$$

TO ROLL SUCH A WF SECTION INTO A 4'-0' DIAMETER SILL, OR SUPPORT ELEMENT, WOULD RESULT IN STRAIN HARDENING OF THE STEEL & VERY LIKELY IN BUCKLING OF THE FLANGES. THEREFORE, THE HATCH SUPPORT FRAME WAS DETAILED FROM PLATES, BARS, & ANGLES TO PROVIDE THE SAME THICKNESSES AND SECTION MODULUS AS THE WIDE FLANGE SECTION.

PIPE COLUMNS

IN ORDER TO SUPPORT THE HATCH ASSEMBLY AND ITS SUPPORT FRAME INDEPENDENT OF THE ENTRANCE PIPE, 4 PIPE COLUMNS ARE USED

$$f_{ALL} \cong 1.65\left(17{,}000 - 0.485\frac{l^2}{r^2}\right) = 28{,}000 - .80\frac{l^2}{r^2} \text{ (Augmented AISC)}$$

$$\text{LOAD/PIPE} = \frac{p \times (\text{SIDE})^2}{4} = 100 \times (3'\text{-}10'')^2 \times 144/4 = 53.5^k$$

$$A_{APPROX} = 53.5/28.0 = 1.92 \text{ SQ. IN.}$$

TRY 3" NOM. DIA. STD. PIPE ($A = 2.228$ SQ. IN., $r = 1.16$ IN)

DESIGN FOR $l = 6$ FT $= 72$ IN (FOR GREATER DEPTH IF REQ'D)

$$f_{ALL} = 28{,}000 - 0.80\left(\frac{72}{1.16}\right)^2 = 28{,}000 - 3{,}100 = 24{,}900 \text{ PSI}$$

$$f_w = 53.5/2.228 = 24.0 \text{ KSI} < 24.9 \text{ KSI} \quad \text{OK}$$

USE 4 - 3" NOM. DIA. STD PIPES TO SUPPORT HATCH SUPPORT FRAME.

TIE BOLT

IN ORDER TO PROVIDE FOR REBOUND OF THE HATCH WHICH MUST BE SECURELY DOGGED TO THE HATCH SUPPORT FRAME, THE SUPPORT FRAME MUST BE TIED DOWN. THIS CAN BE MOST READILY ACCOMPLISHED BY USE OF CABLES OR TIE BOLTS SECURED TO THE HATCH SUPPORT FRAME AND TO THE HORIZONTAL ACCESS PASSAGEWAY BELOW THE VERTICAL ENTRANCE PIPE.

FOR THE DURATION OF THE SIDE-ON PRESSURE AND THE HATCH PERIOD, THE MAXIMUM REBOUND RESISTANCE RANGES BETWEEN 12 PSI AND 30 PSI FOR DUCTILITIES OF 2 AND 5, RESPECTIVELY.

FOR $p_r = 30$ PSI, TOTAL REBOUND FORCE $= p_r A =$

$$30 \times (3'\text{-}10'')^2 \times 144 = 63.2''$$

$$\text{AREA OF TIE BOLT} = \frac{63.2}{4 \times 30} = 0.53 \text{ SQ. IN.}$$

USE 4 - 1" TIE BOLTS ($A = 0.79$ SQ. IN)

Passageway

General concept :

The passageway will connect to the vertical posts in the end bulkhead.

The clear space in the passageway should be at least 3'-0" wide by 6'-0" high.

To circumvent difficulties in forming a transition between the vertical entranceway and the passageway, the passageway will be formed of frames fabricated from WF members. These frames will rest in a sill fabricated from steel members, and they will support timber lagging for walls and roof.

Eliminated in final design because sills and steel covers provide sufficient longitudinal stiffness. { The frames will be fastened to one another by steel pipes acting as struts with threaded rods placed inside the pipes as ties.

The sill for the passageway, to facilitate erection, will be located above the truss in the end bulkhead. At least 6" of earth fill will be required to act as a cushion between the truss and the sill.

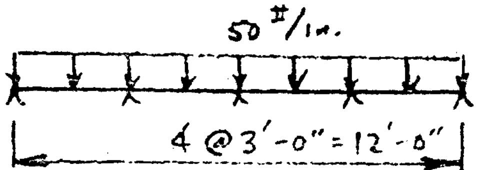
Design of wall sheeting (timber) :

Space steel frames @ 3'-0" on centers longitudinally.

Use lateral force coefficient of 0.5. ∴ loading on sheeting = 0.5 x 100 psi = 50 psi

A module would appear to be reasonable if limited to a 12 ft. length.

Therefore :

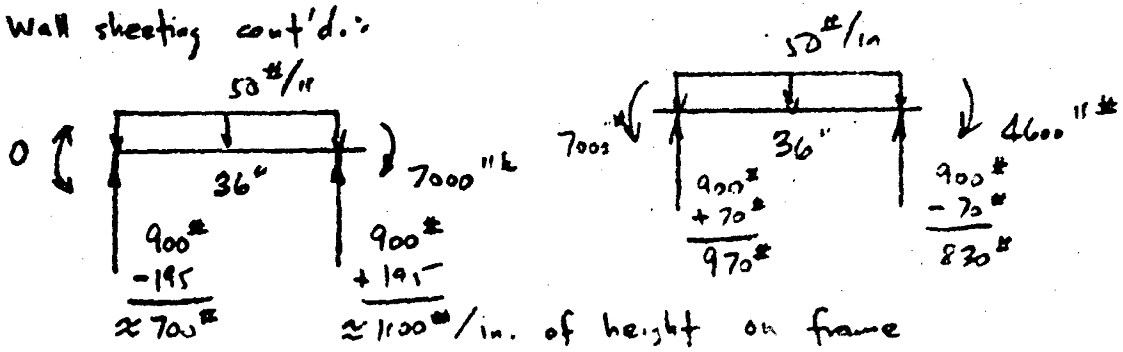


$$\frac{w\ell^2}{12} = \frac{50 \times 36 \times 36}{12} = 5400 \text{ ''\#}$$

0	3/7	4/7	1/2	1/2	
-54	+54	-54	+54	-54	x100 ''#
+54	+27				x100 ''#
	-12	-16	-8	+8	x100 ''#
0	+69	-70	+46	-46	x100 ''#

ROTATION SIGN CONVENTION

Wall sheeting cont'd.:



Max Reaction on Frame = 2070#/in. of height

$M_{max} = 7000$ "#

$$\frac{h^2}{6} = \frac{7000}{7000} = 1$$

$\therefore h = 2.45"$ Try 3x Timber as logging

Check horizontal shear:

$$v = \frac{3}{2}\frac{V}{A} = \frac{3 \times 1100}{2 \times 2.5 \times 1} = 660 \text{ psi} < 700 \text{ ok}$$

Check comp. ⊥ to grain:

Assume flange width = 5"

$$\sigma_{\perp} = \frac{2070}{5 \times 1} = 414 \text{ psi} \quad \text{ok}$$

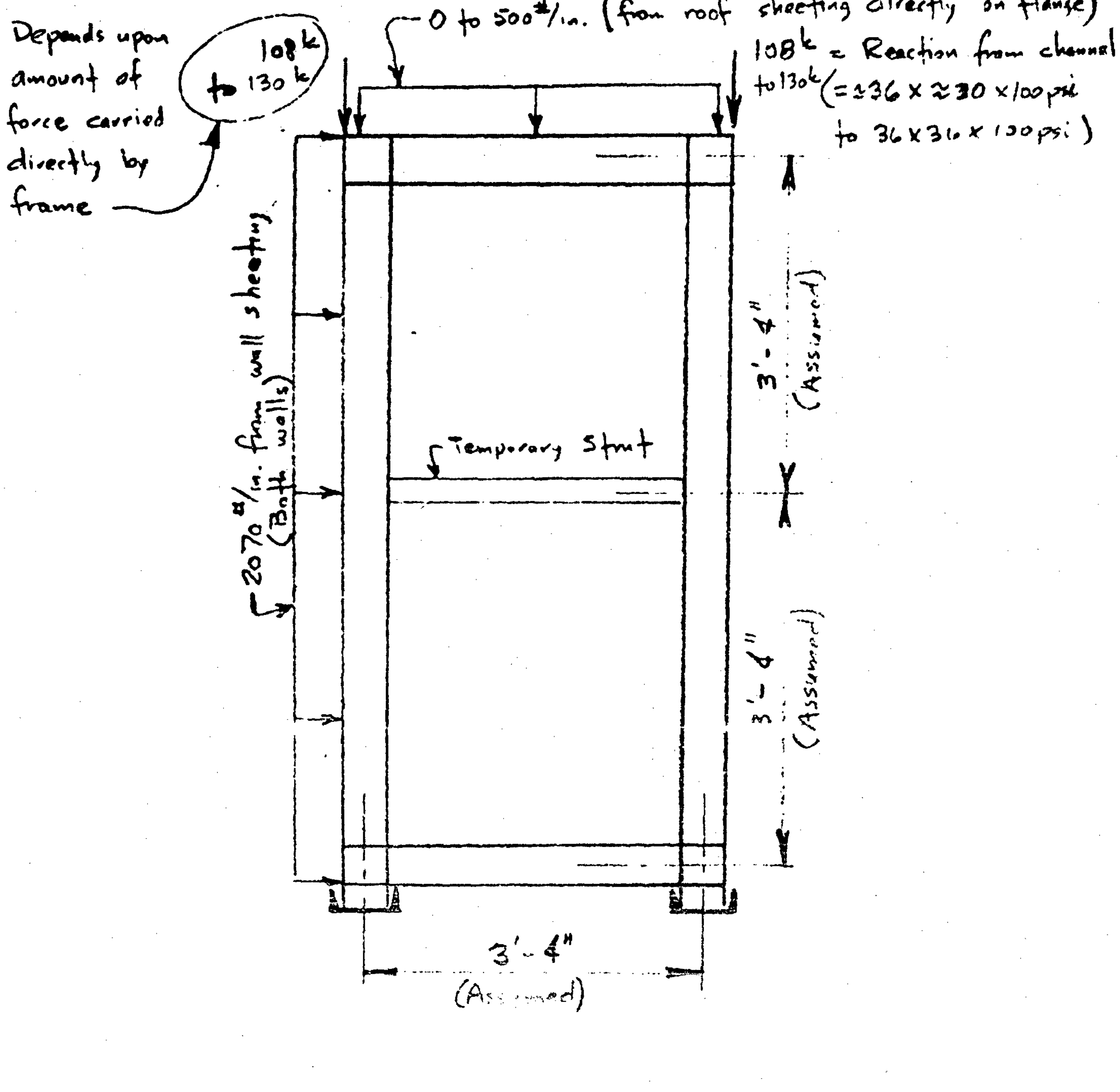
Use 3x (S4S) T&G as sheeting

Frame Design:

Originally the frames were designed to provide a clear height of 6'-0". This design proceeded as shown below, and the 6'-0" height required that an 8WF48 be used in order to fabricate the frame. This made each frame inordinately heavy and other alternates were sought. Finally it was decided that a temporary strut could be placed at the mid-height of the frame. This temporary strut would be installed only during periods of alert.

The timber roofing as originally conceived was to span between steel channels running longitudinally between frames in place of the top most piece of T & G sheeting. This subsequently was replaced by steel covers which span (simple support) directly between the frames. However, the actual design of the frames was accomplished on the basis of the original concept. The later change in concept would not affect the general proportioning of the frames.

Assumed configuration:



In addition to the combinations of loading postulated for the frame, several types of behavior also are possible:

(1) The support condition at the base of the frame considering the influence of the sill could vary from simple support to full fixity.

(2) The frame could be analyzed for "shakedown".

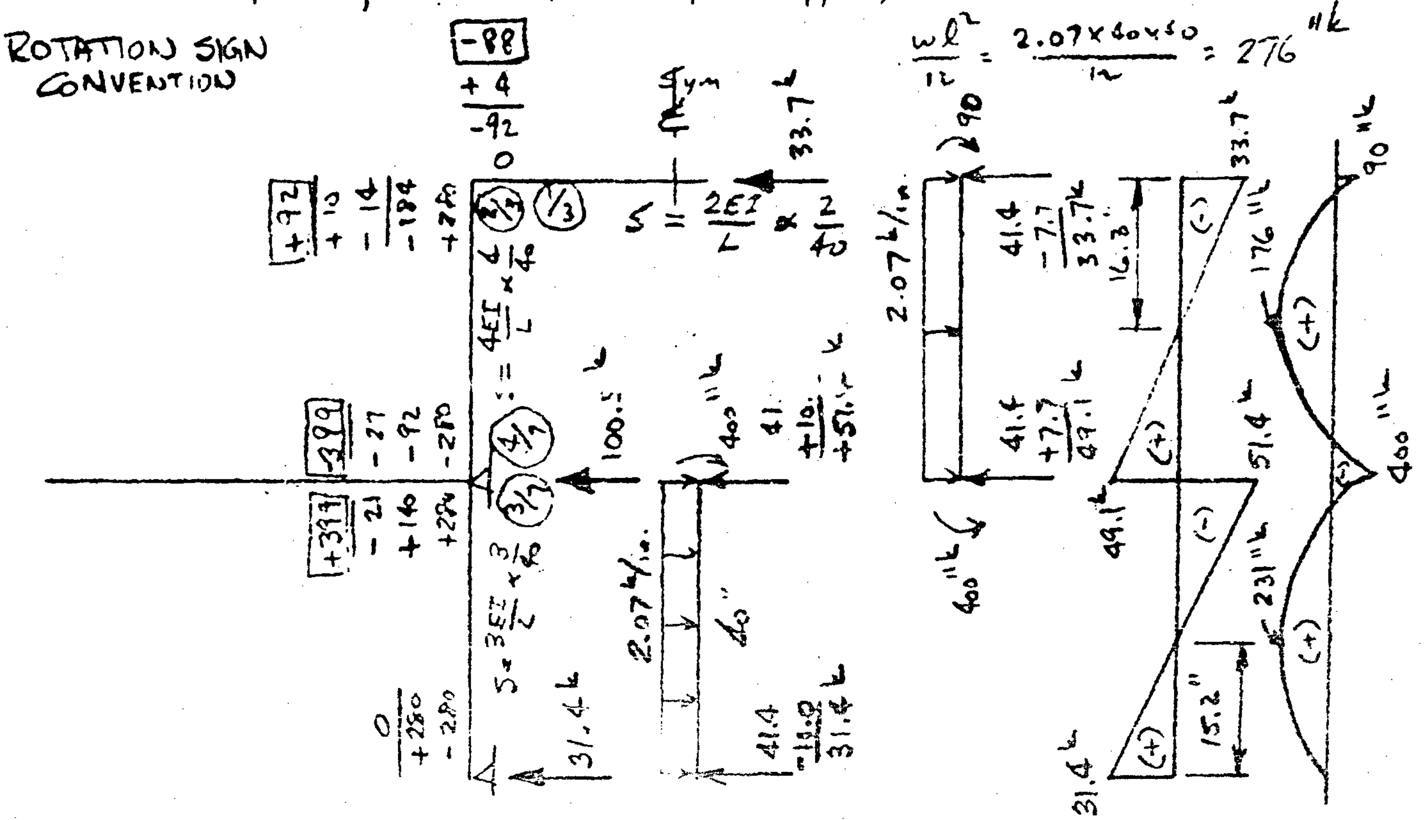
Preliminary analyses (not reproduced here) indicated little difference in the stress distribution for the different loadings postulated. Assuming that the total roof reaction (130k) was applied through the channels attached to the tops of the vertical legs of the frames did indicate slightly larger moments. Therefore, this condition is illustrated below.

Making a "shakedown" analysis and proportioning the frame on this basis seemed unwise since the relatively large deformations needed to form a mechanism in combination with the large axial forces seemed dangerous.

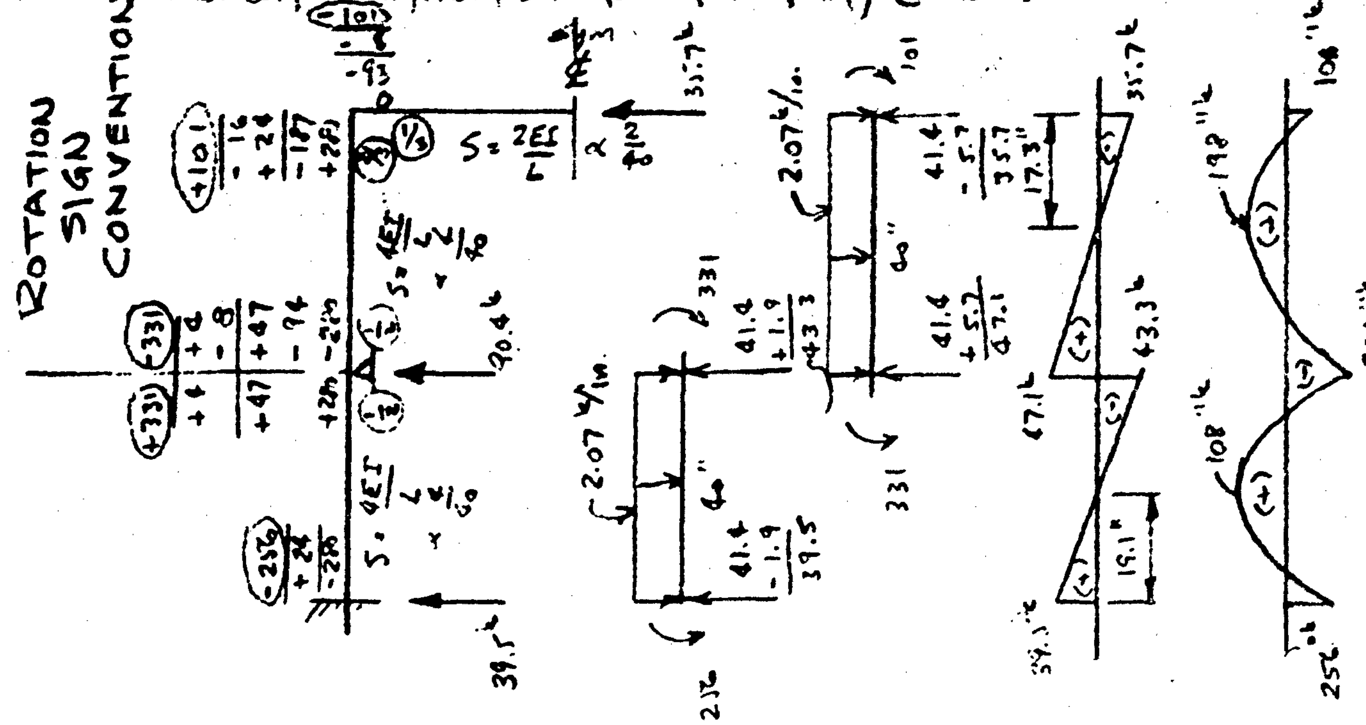
Moment Distribution for simple support @ base:

ROTATION SIGN CONVENTION

$\frac{w\ell^2}{12} = \frac{2.07 \times 40 \times 40}{12} = 276$ "k



Moment Distribution for full fixity @ base:



∴ Max Thrust = 130k (in frame)

Max. Moment = 405''k (in frame)

Max Shear = 51.4k (in frame)

Max Thrust = 101k (in temporary strut)

Proportion Frame:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \sim 1.0 \quad \text{with some conservatism}$$

$$\frac{P}{F_a A} + \frac{Mc}{F_b I} = \ldots$$

Take $F_a = F_b = 45.0$ ksi (Load is more nearly static than dynamic)

$I = Ak^2$ and $k = \sim 0.43\,d$ for WF members.

$$\therefore \; A = \frac{P}{45} + \frac{Md}{45 \times 2 \times (0.43)^2 d^2} = \frac{P}{45} + \frac{M}{16.6d}$$

No 10'' WF section will satisfy since all provide too much area

Proportion Frame cont'd.:

d	A	Section
8"	$\frac{130}{45} + \frac{400}{16.6d} = 5.90\ in^2$	8 WF 20 $\begin{cases} A = 5.88\ in^2 \\ d = 8.14\ in \\ r/d = 0.422 \end{cases}$
6"	$\frac{130}{45} + \frac{400}{16.6d} = 6.89\ in^2$	6 WF 25 $\begin{cases} A = 7.37\ in^2 \\ d = 6.37\ in \\ r/d = 0.422 \end{cases}$

No. 4" WF section will satisfy; none have sufficient area

Try Light beams or joists:

d	A	Section
12"	$\frac{130}{45} + \frac{400}{16.6d} = 4.89\ in^2$	12 B 16.5 $\begin{cases} A = 4.86\ in^2 \\ d = 12.00" \\ r/d = 0.388 \end{cases}$
10"	$\frac{130}{45} + \frac{400}{16.6d} = 5.30\ in^2$	10 B 19 $\begin{cases} A = 5.61\ in^2 \\ d = 10.25" \\ r/d = 0.404 \end{cases}$

Try 12 B 16.5 -- The lightest section:

$$A_{req'd.} = \frac{130}{45} + \frac{400}{90(0.388)^2 \times 12} = 5.35\ in^2 \text{ vs } 4.86\ in^2 \ \underline{NG}$$

Try 10 B 19 -- The next lightest section:

$$A_{req'd.} = \frac{130}{45} + \frac{400}{90(0.404)^2 \times 10.25} = 5.55\ in^2 \text{ vs. } 5.61\ in^2 \ \underline{Ok}$$

However, this section is supplied by only two rolling mills, and it is not a commonly available section. In addition the deep member will require a heavier sill. Thus, try an 8 WF 20, which is only slightly heavier.

$$A_{req'd.} = \frac{130}{45} + \frac{400}{90(0.422)^2 \times 8.14} = 5.95\ in^2 \text{ vs. } 5.88\ in^2 \text{ which}$$

is slightly underdesigned but ok because of slight conservatism inherent in combined stress equation used.

Proportion Frame cont'd.:

Check 8 WF 20:

Buckling:

$L_{?} = 80"$

$r_y = 1.20"$

$L/r_y = 66.6$

$\therefore$ 45 ksi ok

Shear:

$V_{max} = 51.4$

$\frac{V}{A} = \frac{51.4}{0.248 \times 8.14} = 25.4$ ksi ok

Flange buckling:

$\frac{Ld}{bt} = \frac{40 \times 8.14}{5.27 \times 0.378} = 163 < 600 \rightarrow$ 45 ksi ok

Assumed spans ok.

$\therefore$ Use 8WF20 to form the general frame.

Temporary strut:

Max. Thrust $= 101^k$

Assume $F_a = 30$ ksi

$A = \frac{101}{30} = 3.37$ in.^{2}

Try 4WF13 (to take up as little space as possible):

$A = 3.82$ in.^{2}

$r_y = 0.94"$ (Min.)

$L/r_y = \frac{40}{0.94} = 42.6 \therefore$ 30 ksi ok.

Use 4WF13 as support. Note: Detail seat to hold temporary support in place.

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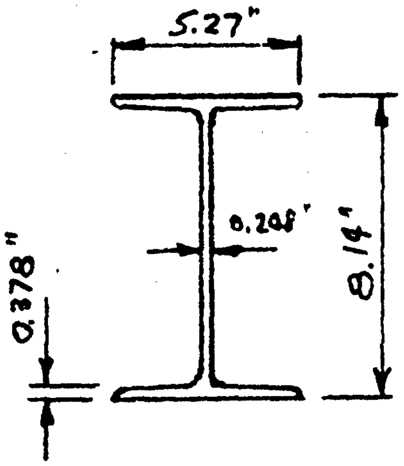
JOB ENG 225
SUBJECT Passageway and Utility Structure Design

SHEET NO. ____ OF 121
BY JLM DATE 3/17/62
CHKD. BY CWG DATE ____

Proportion Sills:

Sills must be dimensioned to accept the vertical member of frames:



Assume the total width of sill will be 12", and compute bearing pressure:

$\frac{130^k}{1 \times 3} = 43.3\ ^k/ft^2$ but shock load (100psi) produces a bearing pressure of $14.4\ ^k/ft^2$ with no structure present. Thus, the net bearing pressure is $28.9\ ^k/ft^2 = 14.5\ ^T/ft^2$. This is quite high in terms of conventional standards but is probably ok if some punching is permitted.

Section modulus req'd. to develop brg. pressure:

Neg. $M_{max} = 6.9''^k$ for $w = 50\,psi \times 12''/ft \times 1'' = 0.6\ ^k/ft$

from wall design

$\therefore$ Neg. $M_{max} = 6.9 \times \frac{43.3}{0.6} = 500''^k$

Pos. M_{max} is much smaller, and a prismatic section is desired. Thus, only neg. moment need be considered.

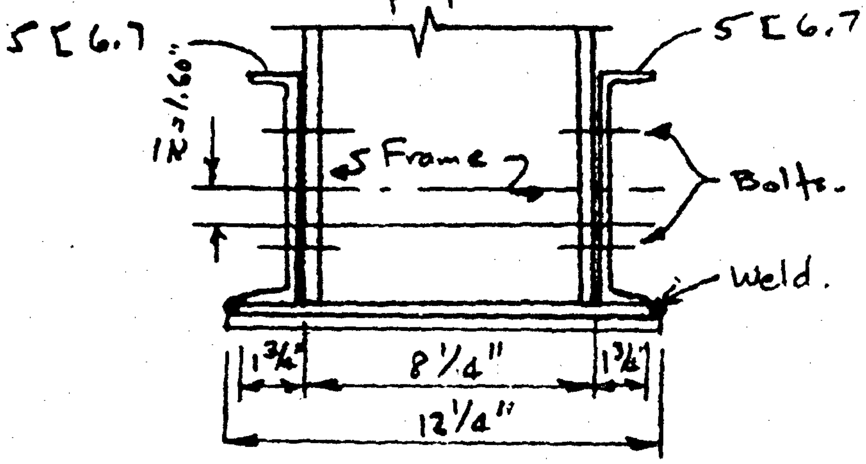
$\frac{I}{c} = \frac{500}{45} = 11.1\ in.^3$

Load more static than dynamic.

Proportion Sills cont'd.:

Several configurations for the sill were investigated:

1. A channel toed up, but no standard channel would suffice.
2. A wide flange beam arranged to resist the moment about its weak axis, but this potentially would cause much difficulty in bedding. Bending about the strong axis caused rather complicated problems of connecting the frames to the sills.
3. A built up section consisting of two channels and a plate:



- Note: the frame forms the splice for the sill since the sill is simply supported at its ends (but continuous on interior).

Several tries were made of the above configuration but the following appeared to provide the lightest section:

2-5][6.7 + $\frac{1}{2}$" x 12 1/4" PL.

	I_{xx}	A	Ax	Ax^2
][	14.8	3.96	0	9.98
PL	0.13	6.13	16.1	8.10
	14.93	10.03	16.1	14.93
			÷ 10.03	43.0 in^4 = I_{cg}
			= 1.60" = $\bar{x}$	

$c = 2.5 + 1.6 = 4.1''$

$\frac{I}{c} = \frac{43.0}{4.1} = 10.5 \text{ in}^3 < 11.1 \text{ in}^3$ but ok since moment considered in design was at center of frame while clear span is ≃ 31".

Weight of Section = 3.42 × 10.03 = 34.3 #/ft.

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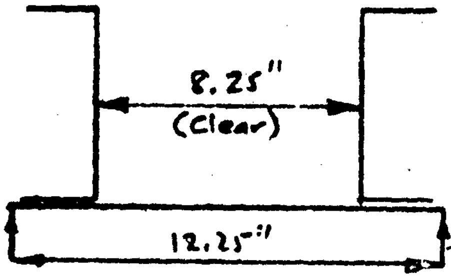
JOB ENG 225
SUBJECT Passageway and Utility Structure Design.

SHEET NO. ____ OF 123
BY JLM DATE 3/17/62
CHKD. BY CNL DATE ____

Proportion Sills cont'd.:

Check stress in bottom ($\frac{1}{2}$") plate as one-way slab:



$\frac{43.3}{12 \times 12.5} = 0.289\ ^{k}/_{in.}$

Allow full plasticity:

$$M_{max} = \frac{0.289 \times \overline{8.25}^2}{16} = 1.23\ ^{''k}$$

$$\frac{I}{c} = \frac{h^2}{6} = \frac{1}{24}$$

$$\sigma = 24 \times 1.23 = 29.5 \text{ ksi} \quad \underline{\underline{ok}}$$

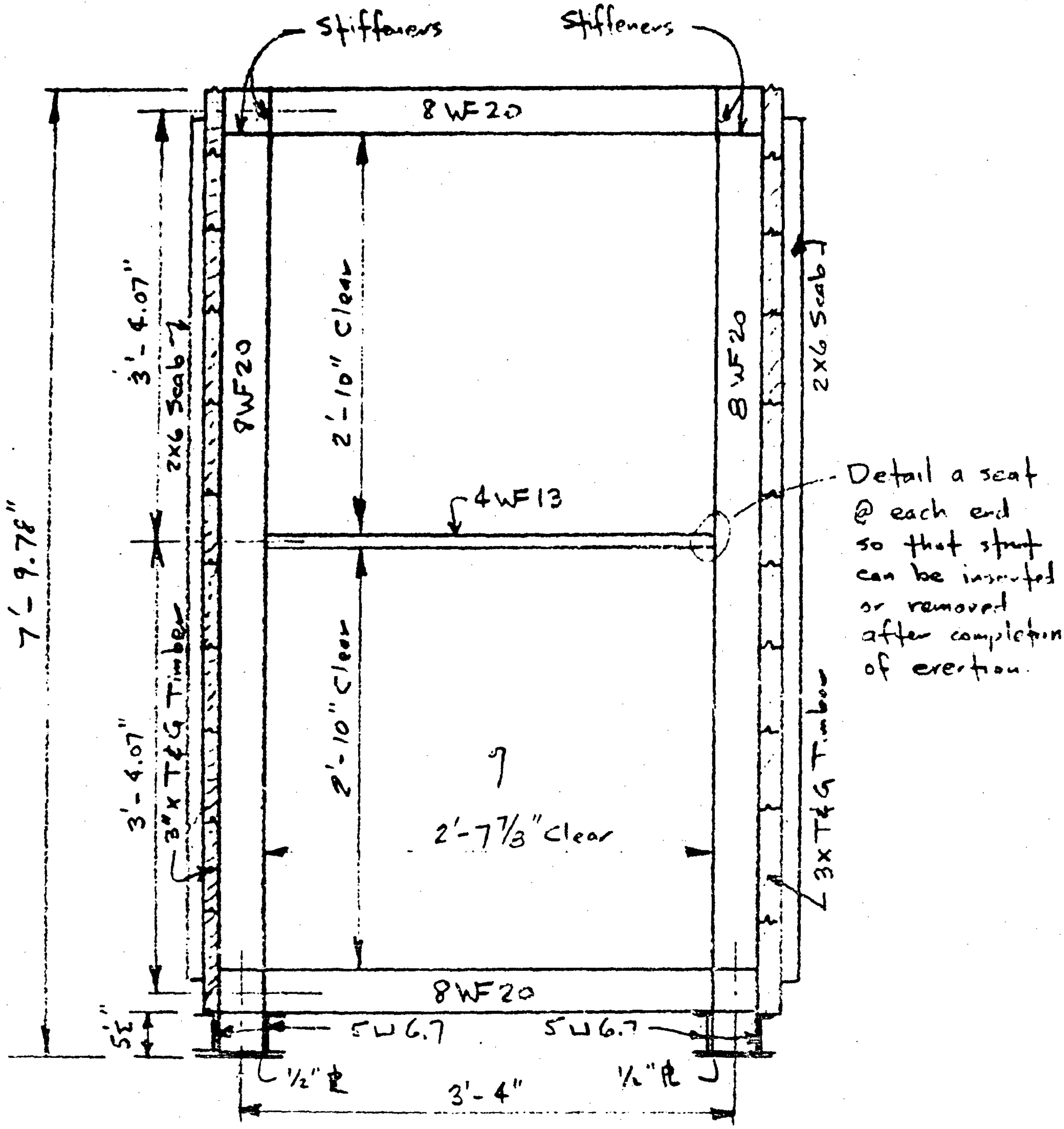
Note that the upper outstanding leg of the channel will form a seat for the T & G timber wall sheeting.

The wall sheeting should be held in place by 2×6 scabs bolted to the steel frames at two locations along the height.
NOTE: Detail a connection for fastening the scab to the steel frames.

The wall sheeting should be placed by loosely bolting bottom of scabs, then placing T & G sheeting between scab and steel frame. After all sheeting is in place, bolt the top of the scab and tighten the bottom scab bolt.

N. M. NEWMARK URBANA, ILLINOIS	JOB ENG 225	SHEET NO. ___ OF 124
	SUBJECT Passageway and Utility Structure Design	BY JLM DATE 3/17/62
		CHKD. BY CWL DATE

Sketch – section through passageway or utility structure:



Proportion roof:

Originally the roof was designed to be constructed of timber which spanned between the wall sheeting. This required 6"x timber which was supported on a 10 ⊔ 20 which replaced the topmost piece of timber wall sheeting. The channel was bolted to each frame by 6 - 3/4" ϕ high strength bolts. The timbers in the roof in turn were held in place by a 6x6 timber laid longitudinally over the top of the roof timbers. A 1⅛" ϕ x13" common bolt placed 3' on centers fastened the 6x6 longitudinal timber to the 10 ⊔ 20.

This system was considered undesirable because of the large amount of heavy timber required and because the high strength bolts used to fasten the channels to the steel sets could easily be confused with common bolts. Furthermore, the placing of bolts through the channel and steel sets would be difficult because of the restricted clearance in the region where the connection was required. -- The stiffeners at the upper corner of the frame caused the main clearance problem.

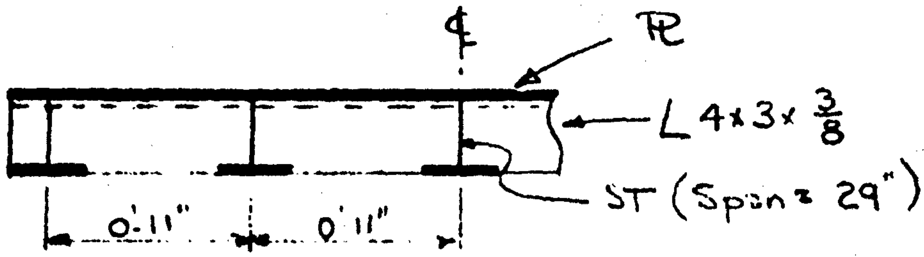
Therefore, steel covers to span (simple support) between steel frames were proportioned to take the place of this timber. This proportioning and the modification needed to make the connection with the ventilation tube are shown on the following two sheets.

The passageway cover providing connection to the vertical entranceway is not actually proportioned. Since, most of the reaction from the hatch cover is carried directly from the hatch support to the steel frames of the passageway by pipe columns, the connection must resist only that force which diffracts around the hatch support. Thus, the force reaching this connection is not know, and a detail of the connection fabricated to accept both the entrance tube and the pipe columns from the hatch support should be adequate to resist whatever force reaches this element.

NOTE: An approximate analysis of this connection as detailed indicates it is capable of resisting a force consistent with a uniform vertical stress of 75 psi.

Design Load = 100 psi

℄ to ℄ Frames - 3'-0"

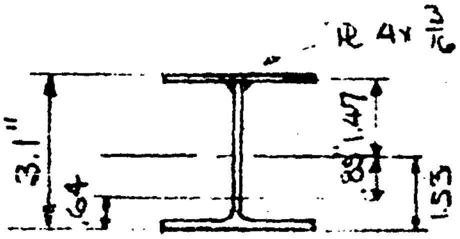


Try $\frac{3}{16}$ PL & ST 3B 4.25

$$Z = \frac{M}{S}$$

$$M = \frac{1}{8} \times 1.1 \times \overline{24}^2 = 116^{\text{"k}}$$

$$Z = \frac{116}{40} = 2.9^{in^3} \text{ Requ}$$



Determine C.G.

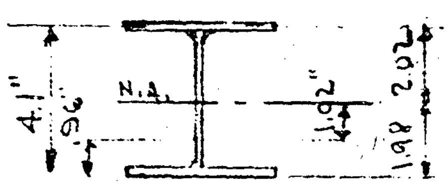
Section	Area	Arm	Moment
PL	.75	3	2.25
T	1.25	.64	.80
	2.00		3.05

$$\bar{X} = \frac{3.05}{2.0} = 1.53"$$

$$I = .90 + 1.25 \times \overline{.89}^2 + .75 \times \overline{1.47}^2 = .9 + 1.1 + 1.6 = 3.6$$

$$Z = \frac{3.6}{1.57} = 2.3^{in^3}$$

Try $\frac{3}{16}$ PL & ST 4B 5



PL	.75	4"	3.00
T	1.48	.96	1.42
	2.23		4.42

$$I = 2.15 + 1.48 \times \overline{1.02}^2 + .75 \times \overline{2.02}^2 \qquad \bar{x} = \frac{4.42}{2.23} = 1.98"$$

$$I = 6.76, \quad Z = \frac{6.76}{2.1} = \boxed{3.2 \text{ Use}}$$

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JOB DA-22-079-eng-225
SUBJECT Cover Design (Cont'd)

SHEET NO. ____ OF 127
BY CWL DATE ____
CHKD. BY JLM DATE ____

Check Horizontal Shear at PL-T Joint

$V = 1.1 \times 15 = 16.5^k$

$Q = .75 \times 2 = 1.5$

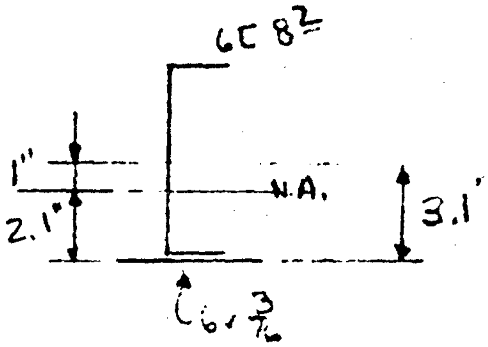
$I = 6.76 \, in^4$

$v = \frac{VQ}{I} = \frac{16.5 \times 1.5}{6.76} = 3.7 \, k/''$

There will be no problem getting enough weld to carry this shear.

Ventilation Structure

[Stiffener on Cover



Section	Area	Arm	Moment
[	2.39	0	0
PL	1.13	3.1	3.5
	3.52		

$\bar{x} = 1''$

$I = 13.0 + 2.39 \times \overline{1}^2 + 1.13 \times \overline{2.1}^2 = 20.4$

$Z = \frac{20.4}{4} = 5.1 \, in^3$

Proportion End Bulkhead:

(Sufficient timber should be supplied in each utility structure to provide an earth bulkhead in the panel between two frames. This bulkhead will act as a fire stop or radiation barrier. A 24" dia. - 14 gage std. corrugated metal pipe should be provided as a crawlway for each earth bulkhead).

Let end bulkhead be wood spanning between vertical legs of frame.

Clear Span = 2'-7.86" = 31.86"

Load = 50 psi = 50#/in of height = 0.05 k/in. of span

$$V_{max} = \frac{0.05 \times 31.86}{2} = 0.795^{k}$$

$$M_{max} = \frac{0.05 \times \overline{31.86}^2}{8} = 2.40''^{k}$$

$$\frac{h^2}{6} = \frac{2.4}{7.0} = 0.342 \text{ in}^2$$

$h = 1.43"$ — Try 2 x timber (Actual h = 1 5/8")

$$v_{max} = 1.5 \times \frac{V}{A} = 1.5 \times \frac{0.795}{1 \times 1.63} = 0.73 \text{ ksi} \quad \underline{ok}$$

<u>Use 2" x timber for end bulkhead</u>

Fasten end bulkhead to steel frame with 2x6 scabs in the same manner as wall sheeting is fastened to frames.

URBANA, ILLINOIS

SUBJECT: Ventilation Tube Design

BY: JLM DATE: 3/31/62

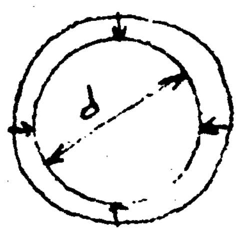
CHKD. BY: CWL DATE:

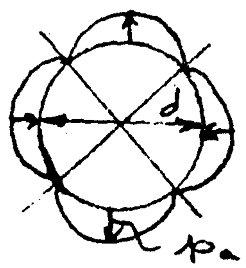
Ventilation Tube:

Investigation of blast values available indicate they are designed for a flow rate of 40 fps. Also a similar flow rate is acceptable to Chemicle Corps CBR filters. Therefor determine duct size on basis of 40 fps flow velocity.

Use same pipe thickness throughout height of tube. Use standard corrugated steel pipe.

Near surface the pipe is subject to the following approximate loads:





$p_u = 50$ psi

$p_a = 50$ psi

$$\sigma_u = \frac{p_u d}{2t}$$

$$\sigma_a = \frac{p_a d^2}{12} \cdot \frac{c}{I}$$

$\frac{c}{I} \cong 0.11t$ for std. Corr. pipe

$$\frac{\sigma_u}{F} + \frac{\sigma_a}{F} \cong 1$$

F = 30 ksi for corrugated steel

$$\frac{p_u d}{2 \times 30 t} + \frac{p_a d^2}{12 \times 30 \times 0.11 t} = 1$$

$p_u = p_a = 0.05$ ksi

$$\therefore t = \frac{0.05}{2 \times 30} d + \frac{0.05}{12 \times 0.11 \times 30} d^2$$

or $t = 0.00083d + 0.00126d^2$

d	0.00083d	0.00126d²	t
8"	0.0066"	0.0808"	0.087" = 12 ga (t = 0.105")
10"	0.0084"	0.126"	0.134" = 10 ga (t = 0.138")
12"	0.0100"	0.182"	0.192" = 5 ga (t = 0.215")
15"	0.0124"	0.284"	0.308" ≥ 1 ga (t = 0.276")*

* 1 ga. = max thickness available.

For larger diameters, no std. corrugated pipe will suffice. However, if the top portion of the pipe is encased in concrete (irrespective of pipe size) a significant saving in material and weight can be realized. Then the pipe need only resist p_u since the concrete will stiffen the pipe sufficiently to allow neglect of flexure near the surface.

Thus

$t = 0.00083d$

Take d = max. = 24"

$t = 0.02''$ → Use 16 ga. pipe ($t = 0.060''$) lightest formed.

Place following table directly on plans:

Air Volume cfm	I.D. Pipe[a] in.	[b] Wall Thickness of Std. Corr. Pipe gage	(Approx. Wt.)	[b] Wall Thickness of Std. Corr. Pipe encased in Conc. gage	(Approx. Wt.)
600	8	12	(12#/ft)	16	(7#/ft)
900	8	12	(12#/ft.)	16	(7#/ft)
1200	10	10	(19#/ft)	16	(9#/ft)
1200-2000	12	5	(37#/ft)	16	(10#/ft)
2000-3000	15	c		16	(13#/ft)
3000-5000	18	c		16	(15#/ft)
5000-6500	21	c		16	(18#/ft)
6500-9000	24	c		16	(20#/ft)

[a] Based on approx. 40 fps intake velocity

[b] Armco Steel & Drainage Products or Equiv. Std. Corrugated Steel pipe (Not Multiplate)

[c] Plain Std. Corrugated Steel plate is not adequate for these cases.

APPENDIX D

RADIATION PROTECTION CALCULATIONS

In this appendix the protection factors for fallout radiation, prompt gamma radiation and neutrons are calculated. Additionally, the prompt gamma doses associated with 100-psi side-on overpressure from surface burst or low air burst are determined.

The protection factor for fallout radiation is calculated in accordance with the procedures contained in OCD Manual "Design and Review of Structures for Protection from Fallout Gamma Radiation," revised 1 October 1961. Since these procedures are available and generally accepted, comments and discussion are omitted and only the calculations are included.

Inasmuch as procedures for calculating protection factors for prompt gamma and prompt neutron radiation are not standardized to the same degree as the fallout gamma radiation, the basic theory and computational procedures are included.

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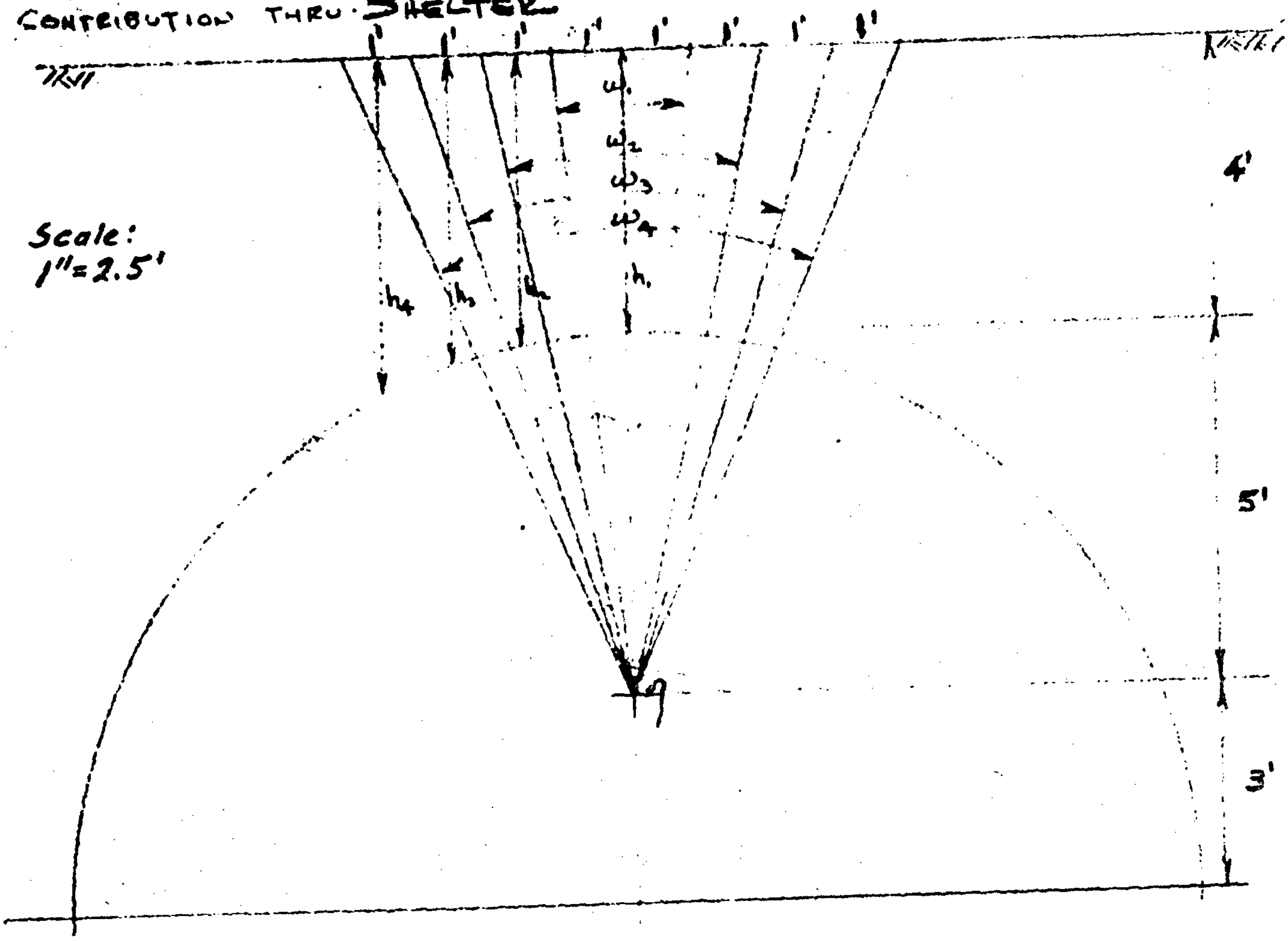
JOB DA-22-079-eng-225
SUBJECT Protection Factor (Fallout)

SHEET NO. ____ OF 132
BY AFD DATE 4-3-62
CHKD. BY ____ DATE ____

CONTRIBUTION THRU SHELTER



h_1 = 4.0' ∴ X_1 = 400 PSF

h_2 = 4.2' X_2 = 420 PSF

h_3 = 4.4' X_3 = 440 PSF

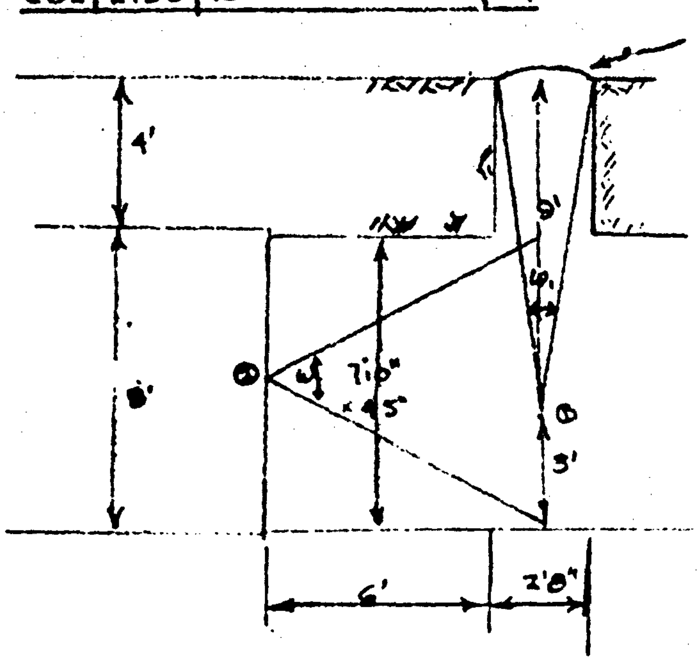
h_4 = 4.8 X_4 = 480 PSF

ω	W	L	Z	e	n	ω	X	$(C_o)_n$ −	$(C_o)_{n-1}$ =	C_o
1	2	45	9	.0445	.4	.06	400			
2	4	45	9	.089	.4	.13	420			
3	6	45	9	.133	.4	.18	440			
4	8	45	9	.178	.4	.25	480			

SO THE OFF CHART 4 OF ENG. MANUAL
NO SENSE TRYING EXTRAPOLATION

P_{f_s} = 20,000 CONSERVATIVE BUT CLOSE

CONTRIBUTION THRU HATCH



$A = \frac{\pi \times \overline{2.67}^2}{4} = 5.58\ sf = \overline{2.37}^2$

$e_1 = 1,\ n_1 = \frac{2\times 9}{2.37} = 7.6,\ \omega = .0085$

SINCE 1/4" STEEL = $\frac{490\ PCF}{1/4 \times 1/12}$ = 10 PSF

ASSUME = 0 & USE CHT. 10

$A_n = .0016 = R_{f①}$

$P_{f①} = 625$

JUST INSIDE SHELTER

$Z = 6 + 1'4" = 7.33$

$W = 4.17'$

$L = 7.83'$

$e = 4.17/7.83 = 0.53$

$n = 2 \times 7.33/7.83 = 1.87$ } $\omega = .085$

$R_{f②} = .0016 \times .085 \times .1 = .0000136$

CONTRIBUTION THRU ARCH

NEGLECT THIS GEOMETRY, $X_0 = 4 \times 100 = 400$ PSF

$C_0 = .000035$

PROTECTION FACTOR

$R_{f②} = C_0 + R_{f②} =$ .000035
.000014
.000049

$P_{f②} = 20{,}000$

IF ASSUMED NO GEOMETRY ABOVE PT ① FOR FRICTION THRU EARTH

$R_{f①} = .0016 + .000035 = .001635$

$R_{f②} = .001635 \times .085 \times .1 = .0000139$

NEGL EFFECT ON P_{f} $P_f = 610$

BASIC THEORY

The uncollided flux received at a point at a distance from a point isotropic source may be expressed as

$$\phi_u = \frac{S_0 \, e^{-\sum_i \mu_i t_i}}{4\pi r^2} \qquad (1)$$

Where S_0 = flux emitted by source in quanta/cm²-sec.
μ_i = linear absorption coefficient for medium i
t_i = thickness of medium i
r = distance from point source

However, since some of the collided flux is scattered rather than absorbed the above expression must be corrected to account for those quanta. This may be done by means of a build-up factor. Further, since the dose may be expressed in terms of quanta, specifically

$$D\left(\frac{r}{hr}\right) = 0.051\,\mu_a^{air}\,E\cdot\phi$$

where μ_a^{air} = linear absorption coefficient (energy absorption) for air
E = energy of the photons
ϕ = total flux at that point

Thus, in general

$$D = \frac{B_i e^{-\sum \mu_i t_i}}{4\pi r^2} \times f(S, E, \mu_a^{air}) \qquad (2)$$

The data furnished on dose (in r) of prompt gamma radiation is literally the integral of equation (2) with respect to time at various distances in air

Specifically

$$D_o = \frac{B_a e^{-\mu_o r}}{4\pi r^2} \int_0^t f(S, E, \mu_a^{o1r})\, dt$$

Inside the structure, assuming the thickness of earth cover is very small compared to r.

$$D_i = \frac{B_a e^{-\mu_o r}}{4\pi r^2} \times B_{(s)} e^{-\mu_s x} \times \int_0^t f(S, E, \mu_a^{o1r})\, dt$$

Defining the protection factor (P_f) to be the dose outside (D_o) divided by the dose inside (D_i)

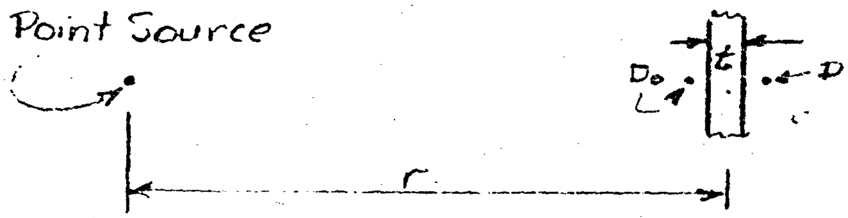
$$P_f = \frac{1}{B_{(s)} e^{-\mu_{(s)} x}} = \frac{D_o}{D_i}$$

Or the attenuation factor, $A_f = \frac{1}{P_f}$;

$$A_f = B_{(s)} e^{-\mu_s x} \qquad (3)$$

where $B_{(s)}$ = the buildup factor for the shielding material
μ_s = the linear absorption coefficient for the material
x = thickness of the material

The solution of the problem would be simple if the thickness of the material were constant and the shield were a plane shield.



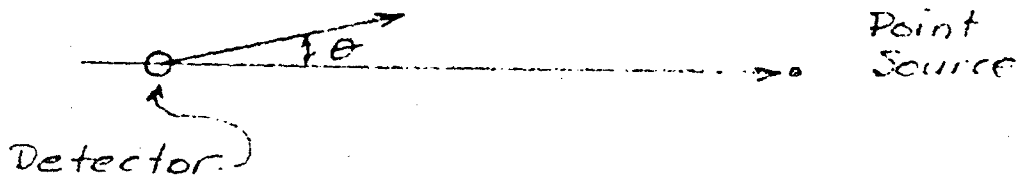
What complicates the problem excessively is the fact that it is not. In fact, equation (3) can be used as a barrier reduction factor only. The geometry of the problem must be handled in a different manner

Spencer[1] investigated the dose angular distribution of radiation from a point source in an infinite homogeneous medium. He used the spectral energy distribution associated with fission product decay at 1.12 hrs after fission. The average energy of these photons is considerably less than the average energy of prompt gamma photons (2).

For Fallout Gamma, $E_a \simeq 1.0$ Mev
For Prompt Gamma, $E_a \simeq 4.0$ Mev.

However, if we use the dose angular distribution obtained by Spencer to determine the dose outside as a function of the problem geometry and the barrier factor from equation (3), the answer obtained should be a reasonable approximation of the true solution.

Specifically, Fig 26.3 of Ref (1) shows the ratio of the total (scattered and unscattered contributions) divided by the scattered contribution as a function of θ for various distances from the point source. This shows that the dose angular distribution doesn't vary significantly with distance from the source.



From Fig 28.6[1] it is apparent that of the total dose received, the scattered component is predominant after a distance of about 429 ft from the point source. For prompt gammas this distance would be somewhat greater, probably on the order of about 850 ft.

In view of the above, it appears reasonable to assume that the dose angular distribution will not vary significantly with the initial energy of the photons and that Figures 28.16 and 28.17 of Ref (1) may be used for the overhead and the surface burst cases respectively. That is, these Figures will be used to obtain the geometry factors for the two cases mentioned.

Note that Fig 28.16 accounts for scattered radiation only. The unscattered component will have to be computed separately. For the surface burst case the only radiation seen by the detector is scatted radiation and the total contribution may be computed as follows:

$$C_p = A_f \cdot G \qquad (4)$$

where C_p = contribution from the point source

A_f = a barrier factor defined by equation (3)

G = a geometry factor obtained from Fig 28.17.

The expression should be conservative since all scattered radiation has lost energy as a result of being scattered. In fact, it can be shown easily that regardless of its initial energy a photon which scatters 90° emerges from the interaction with an energy equivalent to the rest mass of an electron (0.51 Mev) and that which is scattered through an angle of 180° has a residual energy of about half of that (3). Therefore the use of a barrier factor computed for 4.0 Mev photons should be conservative (i.e. result in the computation of a larger contribution than that which would be received in the structure in the field).

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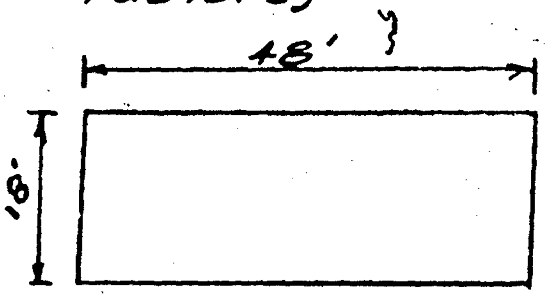
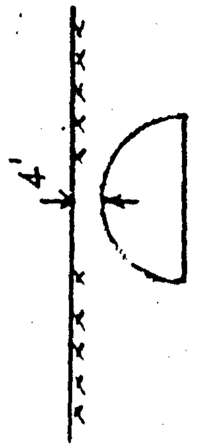
JOB ENGINEERS 225
SUBJECT PROTECTION FROM PROMT GAMMA RADIATION

SHEET NO. ____ OF 138
BY JTH DATE 5/26/62
CHKD. BY AFP DATE ____

COMPUTATION OF PROTECTION FACTOR FOR PROMPT GAMMA - SURFACE BURST

Basic Structure (use main shelter area only for determination of geometry factors)





Contribution From Overhead

1. Thru strip 6' wide with assumed cover of 4' of earth.

a) Geometry Factor

$e = \frac{W}{L} = 6/48 = 0.125$

$n = \frac{2Z}{L} = \frac{2 \times 13}{48} = 0.54$

Then from Chart 3 of Ref 4

$\omega = 0.13$

And from Fig 28.17 of Ref (1)
(Use 1200' Line to be conservative)
$G = 0.06$

b) Barrier Factor

$A_f = B(\mu x)\, e^{-\mu x}$
$= 4.5\, e^{-6.4} = 4.5(0.0017) = 0.00765$

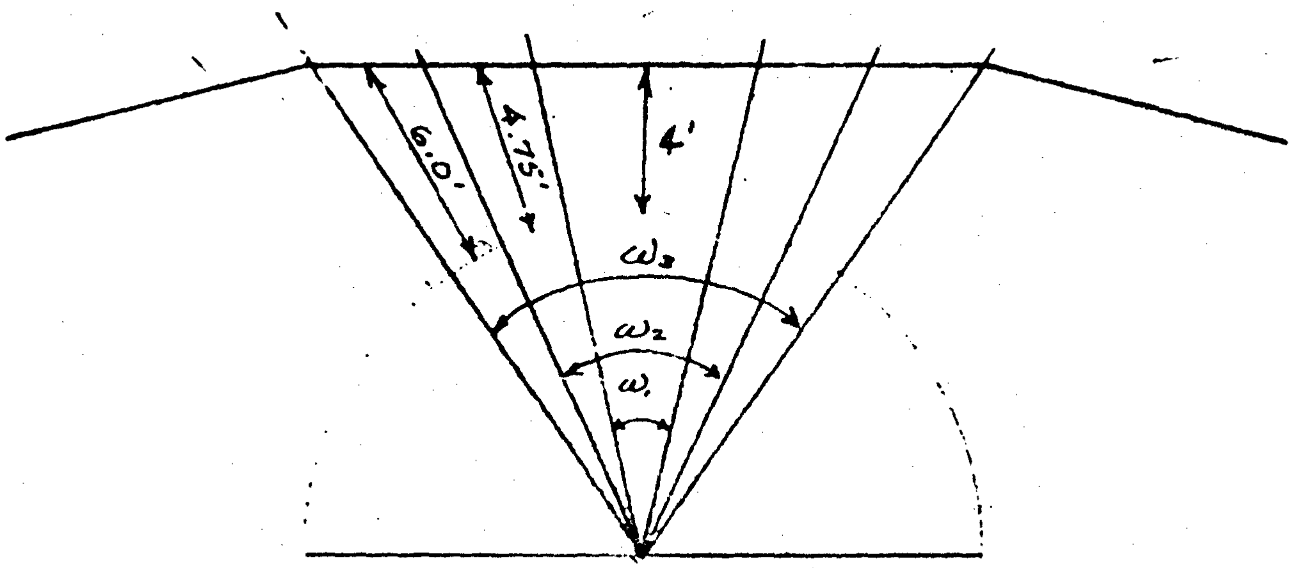
$\therefore \; C_{P_1} = 0.00046$

Note: μ for soil for initial gamma radiation $= \mu_{water} \times \rho_s$
where $\rho_s = 1.6$

For 4.0 Mev; $\mu_s = 0.034 \times 1.6 = 0.0545\ cm^{-1}$

Note: Cont'd
Also, the buildup factor is a function of the energy of the photons and the "distance" μx. Calculations for various Mev photons and for various μx have been tabulated (See Ref (3)).

For 4 Mev photons and $\mu x = 0.0545 \times 48 \times 2.54$ ($\mu x = 6.4$), the buildup factor, $B \doteq 4.5$



DATA TABULATION

ω	W	L	Z	e	n	ω	G	μx	B
ω_1	6	48	13	.125	.54	0.13	.06	6.4	4.5
ω_2	12	48	13	.25	.54	0.25	.12	7.9	5.5
ω_3	18	48	13	.375	.54	0.34	.175	9.97	7.5

$A_{f_2} = 5.5\, e^{-7.9} = 5.5(.00037) = 0.002$

$A_{f_3} = 7.5\, e^{-9.97} \doteq 7.5 \times 4.7 \times 10^{-5} = 0.00035$

$C_{P_2} = A_{f_2}[G_2 - G_1] = 0.002[0.06] = 0.00012$

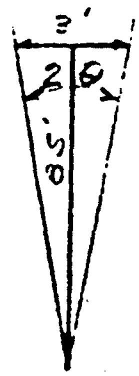
$C_{P_3} = A_{f_3}[G_3 - G_2] = 0.00035[0.055] = 0.000002$

Further calculation is not warranted.

Total:

$C_P = C_{P_1} + C_{P_2} + C_{P_3} = 0.00058$

Contribution Thru Entrance Shaft



For a first approximation assume no shielding by hatch cover.

$\omega = 1 - \cos\theta$

$\cos\theta = \dfrac{8.5}{\sqrt{(1.5)^2+(8.5)^2}} = 0.985$

$\therefore \omega = 0.015$

$G_s < 10^{-2}$ [From Fig 28.17 Ref (1)]

At Shelter Door: Ignore shielding by door

$W = 3 \quad L = 7' \quad Z = 10'$

$e = 0.428 \quad n = 2.86$

From Chart 3 Ref (4); $\omega_2 = 0.035$

$C_s = 0.1\, \omega_2 G_s < 0.000035$

Thus contribution thru entrance shaft is insignificant just inside shelter door even ignoring the shielding afforded by the hatch cover and shelter door. Contributions from other shafts can be neglected also.

PROTECTION FACTOR AGAINST PROMPT GAMMA FROM SURFACE BURST

$P_f = \dfrac{1}{0.0006} = \underline{\underline{1670}}$

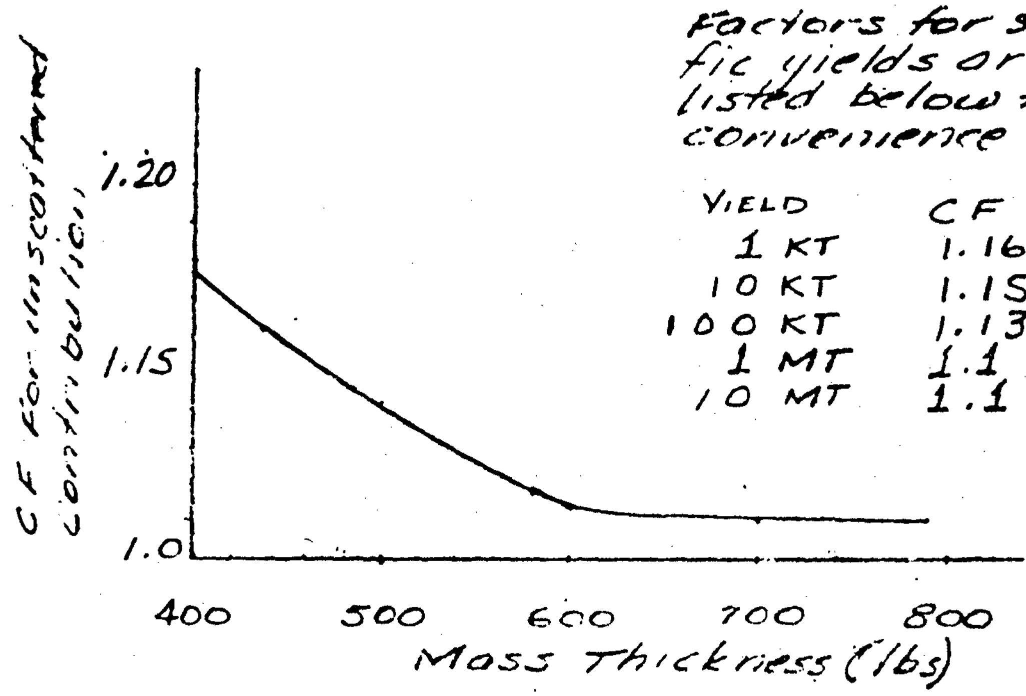
COMPUTATION OF PROTECTION FACTOR FOR PROMPT GAMMA - AIR BURST RIGHT OVER SHELTER

The analytical scheme is the same as for the preceding. The only difference is that the geometry factors G_1 are obtained from Fig 28.16 Ref(1) instead of 28.11. However, it is necessary to correct for the unscattered component. From Fig 28.6 of Ref(1) it may be seen that for fallout gamma photons the components of response due to scattered and unscattered photons vary with the mass thickness of material through which the radiation has passed. The greater the mass thickness, the greater the percentage of the total response due to scattered radiation. A similar relationship is assumed for higher energy photons. It is assumed that the only difference is that a greater mass thickness is required to reduce the proportion of scattered to unscattered components and that the difference is *proportional to the linear absorption coefficients at 1.0 Mev and 4.0 Mev. Since the linear absorption coefficient at 1.0 Mev is about twice that at 4.0 Mev, it should take about twice the mass thickness at 4.0 Mev to produce the same effect.

However the mass thickness involved includes both the mass thickness of the soil overhead and the mass thickness of the air between the structure and the point of detonation. Therefore the correction factor will vary with the weapon yield. It also varies with the solid angle fraction under consideration because the mass thickness varies.

* inversely proportional, that is

To reduce the problem complexity since the majority of the contribution is received through that portion which is 4.0' thick a constant shield mass thickness of 400 lbs will be assumed. To obtain a correction factor, then it will be necessary to add the height of burst divided by 13.3 to 400 lbs and enter the figure below



NOTE: Correction Factors for specific yields are listed below for convenience

YIELD	CF
1 KT	1.16
10 KT	1.15
100 KT	1.13
1 MT	1.1
10 MT	1.1

The correction factor appears to approach 1.1 asymptotically. That is that as the mass thickness increases the total contribution approaches 1.1 × the scattered contribution obtained from Fig. 28.16

From Fig 28.16 Ref(1)

$G_1 = 0.56 \quad G_2 = 0.66 \quad G_3 = 0.71$

Above numbers were obtained using the 3' line, which is conservative, and for values of ω previously computed

Then

$C_{P_1} = A_{f_1} \times C.F \times G_1 = 0.00765[0.56]CF = 0.00428$

$C_{P_2} = A_{f_2} \times C.F \times [G_2 - G_1] = 0.00020$

$C_{P_3} = A_{f_3} \times CF\,[G_3 - G_2] = \underline{0.00002}$

$0.0045\,CF$

Contribution from entrance shaft

$C_S = 0.1\,\omega_2 G_S [C.F.] = 0.1(0.035)(0.215) = \underline{0.00075CF}$

$0.00525\,CF$

Ignore contribution from other shafts and round off

$$P_f = \frac{1}{0.0053CF} = \frac{190}{C.F.}$$

Where CF = a correction factor which is dependent on the weapon yield

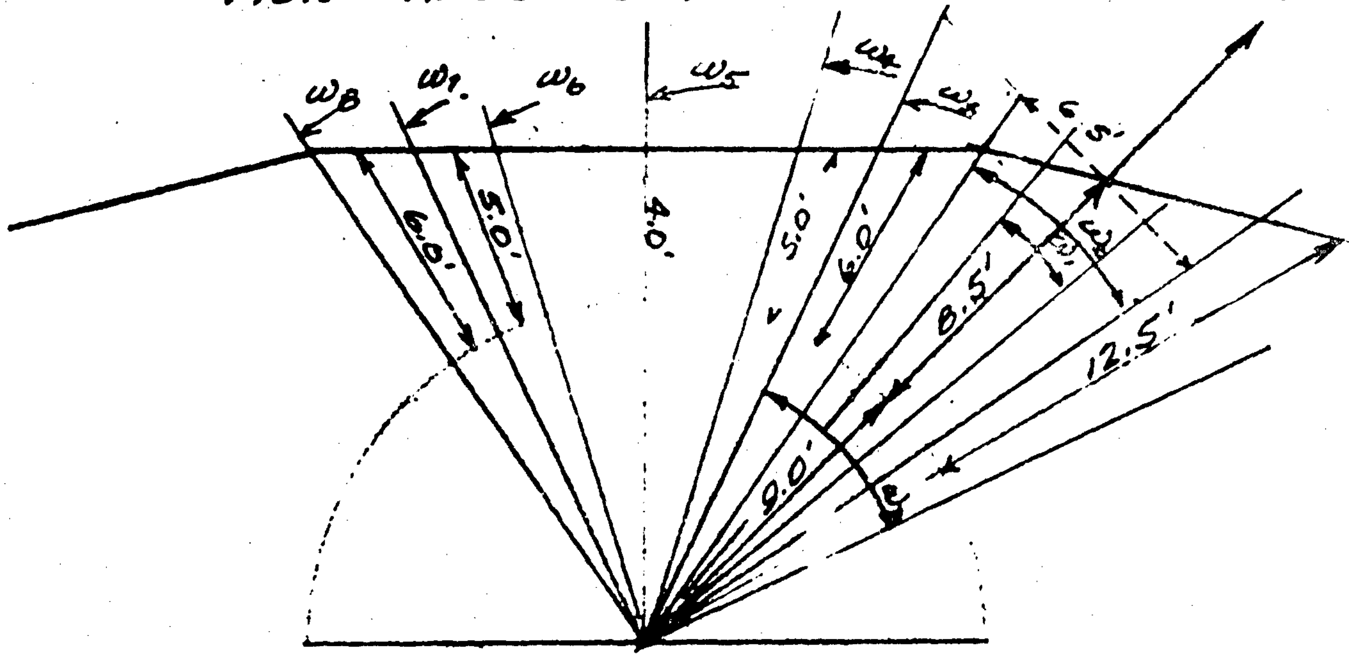
NOTE: The probability that a weapon would be detonated right over the shelter is very small even for a weapon system with a small CEP. The probability may be expressed as

$$p = 1 - 0.5^{\left(\frac{r}{CEP}\right)^2}$$

where r = an equivalent radius for the structure - say its maximum dimension - 10 ft

Then for a weapon system with a CEP of $\overline{48}$ ft the probability of such an event is only 0.5

COMPUTATION OF
PROTECTION FACTOR FOR PROMPT RADIATION - AIR BURST AT 45° TO ONE SIDE



* Geometry factor obtained from Fig 28.16 (1)

ω	W	L	Z	e	n	ω	G*	μx	B	
ω_1	3	48	17.5	.063	.73	.045	.36	14.1	11.5	
ω_2	6.5	48	17.5	.135	.73	.09	.49	14.1	11.5	
ω_3	12.5	48	17.5	.278	.73	.18	.61	9.97	7.5	L
								20.8	21.0	R

$$A_{f_2} = 11.5\, e^{-14.1} = 11.5 \times 7.5 \times 10^{-7} = 8.6 \times 10^{-6}$$

$$A_{f_{3L}} = 7.5\, e^{-9.97} \Big\} = 0.00035$$

$$A_{f_{3R}} = 21\, e^{-20.8} = 21 \times 9 \times 10^{-10} = 1.89 \times 10^{-8}$$

$$C_{P_2} = A_{f_2} G_2\, C.F. = 8.6 \times 10^{-6} \times 0.49 = 4.23 \times 10^{-6} \times C.F.$$

$$C_{P_3} = \tfrac{1}{2} A_{f_{3L}} [G_3 - G_2] = \text{Negligible}$$

$$+ \tfrac{1}{2} A_{f_{3R}} [G_3 - G_2]_A^{CF} = 0.5 \times 3.5 \times 10^{-4} \times 0.11 = 19.2 \times 10^{-6} \times C.F$$

Try one more on left: use W = 18.5 ; e = .386, n = .73
ω_4 = 0.25 and G = 0.66

$$C_{P_4} = \tfrac{1}{2} A_{f_{4R}} [G_4 - G_3] = 0.5 \times 2 \times 10^{-3} [0.05] C.F = 50 \times 10^{-6} \times CF$$

5. For ω_5 ; $W = 35'$ $L = 48'$ $Z = 17.5'$

$e = .73$, $n = .73$ and $\omega_5 = 0.38$

Then $G_5 = 0.73$. & Assume $X = 48''$ $A_{f_{5L}} = 7.65 \times 10^{-3}$

$C_{P_5} = \frac{1}{2} A_{f_{5L}} [G_5 - G_4] C.F. = 0.5 \times 7.65 \times 10^{-3} [0.07] CF$

$C_{P_5} = 268 \times 10^{-6} \times CF$

6. For ω_6 ; $W = 48'$ $L = 66'$ $Z = 17.5'$

$e = 0.73$ $n = 0.53$ and $\omega_6 = 0.50$

Then $G_6 = 0.78$. Assume $x = 48''$; $A_{f_{6L}} = 7.65 \times 10^{-3}$

$C_{P_6} = \frac{1}{2} A_{f_{6L}} [G_6 - G_5] C.F = 0.5 \times 7.65 \times 10^{-3} [0.05] CF$

$C_P = 192 \times 10^{-6} \times CF$

7. For ω_7 ; $W = 48'$, $L = 2 \times 17.5 \tan 71° = 103'$, $Z = 17.5'$

$e = .466$, $n = .34$ and $\omega_7 = 0.56$

$G_7 = 0.8$.. Assume $x = 60''$ $A_{f_{7L}} = 2 \times 10^{-3}$

$C_{P_7} = \frac{1}{2} A_{f_{7L}} [G_7 - G_6] CF = 0.5 \times 2 \times 10^{-3} [0.02] CF$

$C_{P_7} = 20 \times 10^{-6}$ CF

8. For ω_8 ; $W = 48'$, $L = 2 \times 17.5 \tan 80° = 198.5'$, $Z = 17.5'$

$e = 0.242$, $n = .176$ and $\omega_8 = 0.60$

$G_8 = 0.81$. Assume $x = 72''$, $A_{f_{8L}} = 3.5 \times 10^{-4}$

$C_{P_8} = \frac{1}{2} A_{f_{8L}} [G_8 - G_7] CF = 0.5 \times 3.5 \times 10^{-4} [0.01] CF$

$C_{P_8} = 1.75 \times 10^{-6}$

TOTAL = $C_P = 555.2 \times 10^{-6}$ CF

$P_f = \frac{1800}{C.F}$ Say 1550 for CF = 1.16

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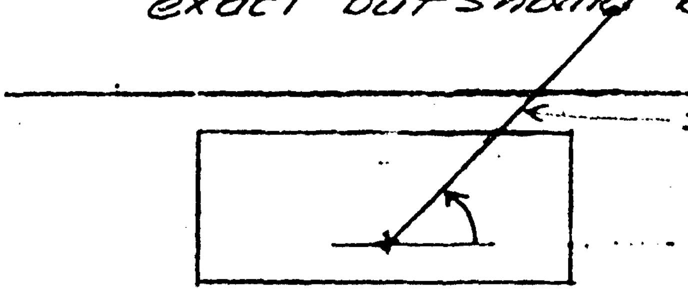
JOB ENGINEERS 225
SUBJECT PROTECTION FROM PROMPT GAMMA RADIATION

SHEET NO. ____ OF 146
BY JTH DATE 5/28/62
CHKD. BY AFD DATE ____

COMPUTATION OF PROTECTION FACTOR FOR PROMPT GAMMA RADIATION – AIR BURST AT 45° OFF ONE END

Use same analytical scheme as for Surface burst except multiply all cover thicknesses by $\sqrt{2}$. This is not exact but should be in the ball park.



secant thickness $= h\sqrt{2}$

$x_1 = 48\sqrt{2} = 68''$
$x_2 = 57\sqrt{2} = 80.5''$
$x_3 = 70\sqrt{2} = 99''$
$x_4 = 88\sqrt{2} = 127''$

ω	W	L	Z	e	n	ω	G^*	μx	B
ω_1	6	48	13	.125	.54	.13	.56	9.06	6.5
ω_2	12	48	13	.25	.54	.25	.66	10.74	8.0
ω_3	18	48	13	.375	.54	.34	.72	13.2	10.0
ω_4	24	48	13	0.50	.54	.50	.78	17.6	16.0

$A_{f_1} = 6.5e^{-9.06} = 6.5 \times 1.18 \times 10^{-4} = 7.67 \times 10^{-4}$

$A_{f_2} = 8.0e^{-10.74} = 8.0 \times 2.2 \times 10^{-5} = 1.72 \times 10^{-4}$

$A_{f_3} = 10e^{-13.2} = 1.7 \times 10^{-5}$

$A_{f_4} = 16e^{-17.6} = 16 \times 2.3 \times 10^{-8} = .368 \times 10^{-6}$

$C_{P_1} = A_{f_1} G_1 [CF] = 7.67 \times 10^{-4} \times 0.56 \times CF =$ 4.[illegible] $\times 10^{-4} \times CF$

$C_{P_2} = A_{f_2}[G_2 - G_1]CF = 1.72 \times 10^{-4} \times 0.10 \times CF = 0.17 \times 10^{-4} \times CF$

$C_{P_3} = A_{f_3}[G_3 - G_2]CF = 1.7 \times 10^{-5} \times 0.06 \times CF =$ Negligible

Total $= 4.46 \times 10^{-4}$ CF

$$Pf = \frac{2200}{CF} \simeq 1900$$

This was not corrected for contribution thru entrance shaft

COMPUTATION OF PROTECTION FACTOR FOR PROMPT NEUTRON RADIATION

Unfortunately, to my knowledge there has been no computation of the dose angular distribution of neutrons. Such a computation is beyond the scope of this work. Therefore, we will obtain a barrier factor which is a lower bound for the protection factor since the geometry factor is certain to be less than 1.

From Article 8.64 of Ref(2), a tenth value layer for damp earth is 15". Using an average mass thickness of 6' = 72", the barrier factor can be computed to be $(0.1)^5 = 10^{-5}$. This should be sufficient to reduce the neutron flux to an insignificant level and to shield against the capture gammas resulting from the capture of thermal neutrons.

Note that even if the minimum thickness over head is used the protection factor afforded is greater than 1000 without taking the geometry factor into consideration

Under these conditions it will be assumed that if the structure provides adequate protection against prompt gamma it will provide adequate protection against the prompt neutron dose.

PROMPT GAMMA DOSE ASSOCIATED WITH 100 PSI SIDE-ON FROM SURFACE BURST OR LOW AIR BURST

YIELD	RANGE OF 100 psi (ft)	ASSOCIATED GAMMA (R)	DOSE INSIDE (R)	
1 KT	350	73,000	44	
10 KT	754	253,000	152	
100 KT	1630	310,000	186	Peak
1 MT	3500	203,000	122	
10 MT	7540	22,800	14	

For higher yield weapons, the dose inside will be less. To reduce prompt gamma dose to less than 50 R add two half thicknesses of cover or 15".

The protection factors computed are undoubtedly conservative for reasons previously discussed. However, to correct the the condition costs very little. So it is probably not unreasonable to specify 5 ft of cover rather than 4.

REFERENCES

(1) Spencer, L.V.
"Structure Shielding Against Fallout Radiation from Nuclear Weapons"
March 1961 (Unpublished)

(2) Defense Atomic Support Agency
"The Effects of Nuclear Weapons"

(3) Price, B.T. et al
"Radiation Shielding" Pergamon Press, New York, 1957

(4) FitzSimons, L.N.
"Design and Review of Structures for Protection from Fallout Gamma Radiation" Revised 1 Oct 1961
(Unpublished)

APPENDIX E

ALTERNATE ENTRANCE STRUCTURES

While the amended contract called only for a vertical personnel entrance structure, the original contract called for consideration of other types of entrance structures including both personnel and vehicular entranceways. Since some preliminary effort was directed toward the consideration of entrance structures other than the vertical type, this appendix has been included to discuss briefly the factors affecting the design of entrance structures. Consideration of this phase of the study did not proceed sufficiently to provide quantitative conclusions. Thus, details are not considered here. Since Phase II of this contract will be devoted specifically to the development of entrance structures, the discussion presented now should be considered as tentative and subject to revision.

In general the major factors controlling the selection of a type of entranceway structure and door are as follows:

1. Capacity (size required)
2. Structural Resistance
3. Radiation Protection
4. Cost
5. Ease of Erection
6. Simplicity and Positiveness of Operation
7. Effect on Shelter Proper
8. Vulnerability to Damage or Blocking

Preliminary consideration appears to indicate that it will be more economical to design single file entrance doors than for multiple files. This is due to the fact that the cost of the door (or the intervening jambs) increases with the width to be spanned.

In determining the required structural resistance of a door to resist a specific overpressure, the angle that the door makes with the horizontal ground surface must be considered. If the door is flush with the ground surface it may be designed for the overpressure only. If the door is at right angles to the horizontal ground surface, and thus perpendicular to the oncoming blast it must be designed for a significant multiple of the overpressure, the reflected pressure. Doors at intermediate angles can be designed for pressures between one and several times the overpressure depending on the angle of incidence and the magnitude of the overpressure.

It does not appear feasible to design doors to serve as radiation barriers per se. In general a more economical solution appears to result when bends or turns are placed in the entranceway, or by placing a barrier or shield inside the shelter.

Ease of erection was a primary consideration of this study. This in itself limits the type of door that could be considered. Not only must the weight of the door be considered, but the weight of the jamb and supporting members as well. For the specific structure considered in this report it was necessary that the support for the door be isolated from the shelter to prevent the door from inducing loads directly upon the shelter.

The ideal type of door would be simple to operate, yet would make an absolute seal. These conditions tend to militate against each other.

If the door is heavy some auxiliary mechanism must be used to close and open the door. Such a mechanical device may take time to operate and thereby slow the entire "buttoning up" process.

Assuming that an entrance structure is to be designed for a given width (lesser dimension) and overpressure the total cost will depend primarily upon the length of the door and the length of the passageway or tunnel. In general the length of the door and the length of the passageway are functions of the secant of the angle between the door and the entranceway floor. The effect of the change in the angle between the door and the entranceway floor is clearly evident in Fig. E.1. The 1:10 slope represents about the maximum safe slope for a personnel ramp, while the slope of the stairs is equivalent to a 7 3/4" riser and 9 1/2" tread. The latter represents about the maximum stair slope.

Fig. E.1(a) indicates that it is not reasonable to use a ramp and a horizontal door. Both the door and the entrance tunnel are too long to be economical. A much more reasonable length of door and entranceway tunnel can be achieved by using a cut and cover method of construction and by placing the door at some angle less than about 45° with the horizontal as shown in Fig. E.1(b). An alternative to the cut and cover method is to use a submerged entranceway placing the door below the ground level on the side of the excavation as shown in Fig. E.1(c). The latter case has the obvious disadvantage that the excavation may serve as a basin to collect surface runoff, although this problem could be lessened by placing the bottom of the excavation lower than the entrance floor level. The entrance tunnel shown in (b) may be shortened by placing more cover over the shelter. Thus the cost of the entrance tunnel must be weighed against

the cost of the additional fill or cover, considering both the radiation through the shelter roof and through the entranceway.

Figure E.1(d) and (e) compare the lengths of door and of entrance structure for a horizontal door and a door on the side slope of a cut and fill construction, when a stairway is used. The difference in cost in this case, as far as the door and entrance tunnel only are concerned, is apparently little due to the fact that the door in (e) must be designed for a reflected pressure whereas the horizontal door need be designed only for the overpressure. When the total cost of the overall structure itself is considered, the cut and cover method in (e) will be generally less expensive since the original excavation required for the shelter itself is less than that required for (d).

The length of the door required in Fig. E.1(a) can be shortened considerably by leaving the actual entranceway open and by placing the door in the interior of the entranceway tunnel. The location of a door in the entrance tunnel proper has three major disadvantages; (1) the door must be designed for the full reflected pressure, p_r, of approximately five times the overpressure of 100 psi; (2) it requires a separate and independent structure to accommodate the door; and (3) the foundation for the door to resist this higher pressure is more difficult to fabricate.

The latter two disadvantages may be partially resolved by placing the interior door adjacent to the shelter proper. This eliminates the need for a break in the entrance structure and simplifies the design of a door jamb or foundation to resist the total reflected pressure. It poses an additional problem, however, as the shelter wall itself must now be significantly strengthened.

If the door is placed at the exposed end of the entrance tunnel, the tunnel itself must be designed to minimize the possibility of collapse due to the longitudinal pressure along the tunnel resulting from the blast pressure on the door. This may be achieved by placing the heavy jamb around the entrance tunnel to transmit the load from the door to the soil itself and by using a somewhat larger cross section of entrance tunnel adjacent to the door than that used in the interior of the entrance tunnel. The larger section of the entrance tunnel will slide (telescope) sufficiently to prevent the full longitudinal force from being applied to the remainder of the entranceway tunnel.

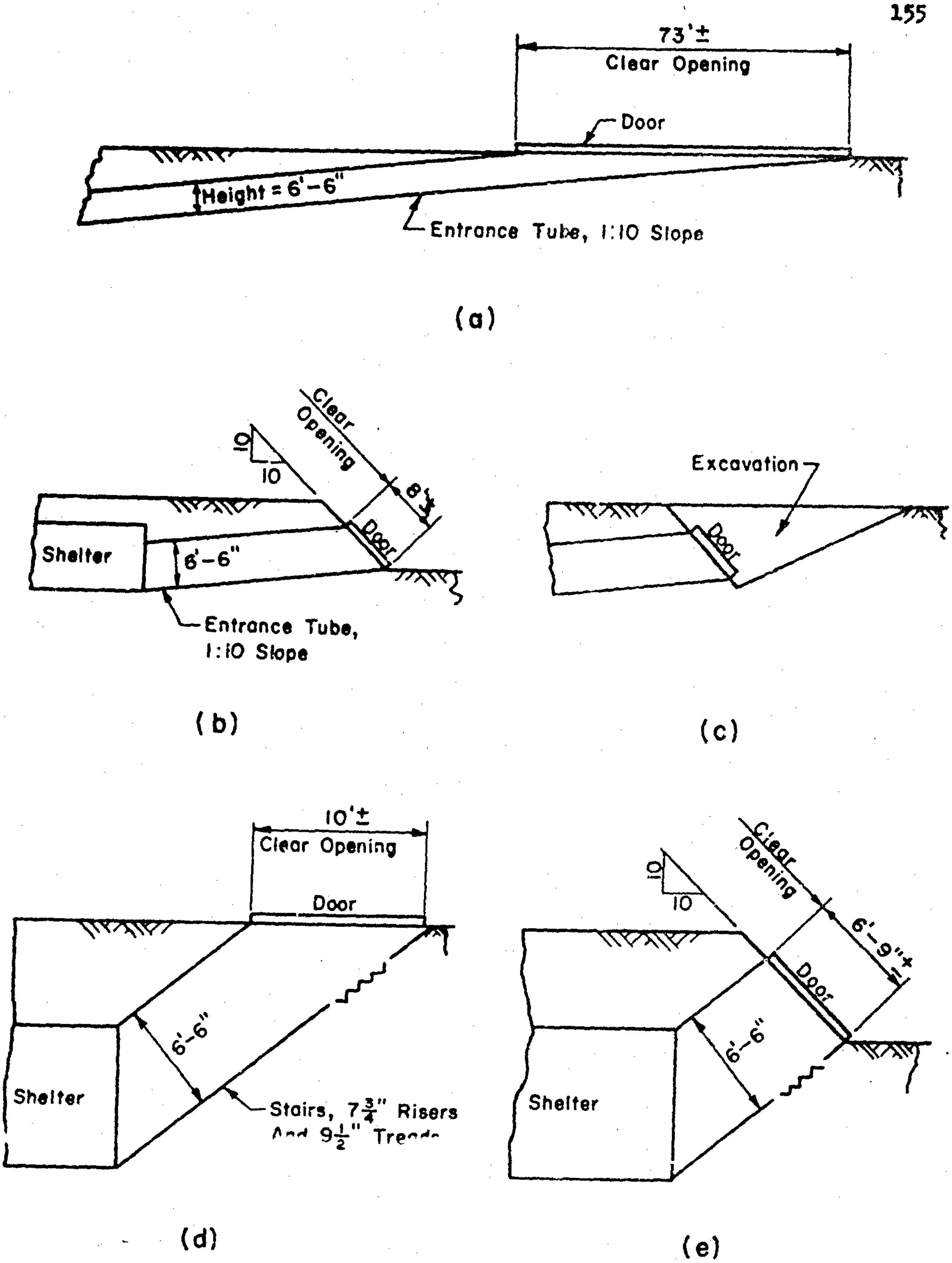


FIG. E.1 EFFECT OF ENTRANCE STRUCTURE GEOMETRY.

APPENDIX F

BEHAVIOR OF UNDERGROUND ARCHED STRUCTURES UNDER BLAST LOADING

INTRODUCTION

This appendix includes a study of the behavior of completely buried arches and conduits under the effects of full scale nuclear detonations. To date a relatively minor amount of information pertaining to this particular type of construction exists. However, the use of such construction now is recognized as an excellent means of providing economical protection against blast phenomena. As a result the knowledge of the behavior of completely buried arch structures must be developed.

The first subject considered in this appendix is an evaluation of the design criteria presented in Ref. (F.1)*. This evaluation involves a comparison of the observed results obtained in Operations Plumbbob and Hardtack with computed results using the currently available criteria. It was obvious from this comparison, especially when the computed results were extended to include larger weapon yields, that although the existing design procedure was simple, it generally yielded results which were too conservative. That is, the computations indicate a larger resistance must be provided in the structure than was evidenced by the behavior of these structures under actual test conditions. Therefore, it was necessary to modify the existing design procedure to take account of the observed behavior.

*Numbers in parentheses refer to the corresponding entry in the List of References for this appendix.

It was conceived that the conservatism of the existing criteria might result from the implicit assumption that the foundations did not move, an assumption which yields an upper bound solution for structures in soil. For any structure founded in soil it is obvious that some motion of the footings must occur. This motion would of course reduce the forces acting along the upper surface of either an arched or rectangular structure. A method of accounting for this footing motion is developed in this appendix, and this method appears to give net pressures acting on the structure which are on the safe side when the results of this method are compared with the few free field measurements which are available. The derivation of this procedure is described completely herein.

Finally, strength predictions for the buried arches and conduits which were tested in Operations Plumbbob and Hardtack based upon material properties for these structures are presented. These strength predictions are compared with the actual test experience, the computed results obtained by a consideration of the "upper bound" solution, and the computed results for a hypothetical weapon of large megaton yield.

RESULTS OF "UPPER BOUND" SOLUTION

The assumption, implicit in the procedure given in Ref. (F.1), that the footings of the structure are immovable constitutes an upper bound since the bearing pressure developed under the footing will deform the soil to an extent which depends upon the size of the footing and the forces acting on the structure. Generally completely buried construction is economically feasible for relatively high overpressures. The exact overpressure where complete burial becomes feasible depends on many factors which must be evaluated for each particular site. However, it appears likely that

underground protection should be seriously considered for overpressures in excess of 30 psi. Data are relatively non-existent for the displacement imparted to the soil by the air-burst of a nuclear weapon, but in Shot Priscilla of Operation Plumbbob, a transient displacement of the ground surface of approximately 3 in. was observed at the 125 psi overpressure level. This measurement was obtained where there were no structures buried below the ground surface. It is obvious that had there been a structure located flush with the ground surface at the point of this measurement, and supported by footings smaller than the roof area, the transient displacement of the structure would have exceeded the 3 in. actually recorded.

Despite this measurement from which significant motion of the structure could be inferred, it seemed desirable, for the sake of simplicity and for developing a quantitative feeling for the problem, to assume that no vertical motion of the structures would occur. Furthermore, if the bounds to the solution of a problem are separated by an amount which is comparable to the degree of accuracy to which the input data can be specified there is no justification for considering more than these bounds. Accordingly the criteria specified in Ref. (F.1) were applied directly to the buried arches and conduits tested in Operations Plumbbob and Hardtack. Since all of these structures fulfilled the criterion for complete burial only the direct compression mode of loading (the mode causing only hoop compression in the arch rib) was considered. The peak overpressure acting at the ground surface which would cause <u>failure</u> by general yielding (assumed to be defined by $\mu = 5$ for reinforced concrete and by $\mu = 20$ for steel) of the arch rib to be imminent was computed for weapon yields ranging from moderate kiloton to large megaton values. The results of this computation

for the smallest and largest yields considered are shown in Table F.1 under the column entitled Footings Immovable. These results are slightly in error because the overpressures for the steel structures have been modified from the original computations by multiplying by the ratio of the probable yield strength of 40 ksi to the assumed yield strength of 60 ksi. This modification is not rigorously correct since a change in overpressure would necessitate generally a minor change in the effective duration of the forcing function. However, the latter effect was neglected since it would not affect seriously the relative magnitudes of the values or the conclusion derived from them. The necessity for making this yield strength correction resulted from the fact that at the time the original computations were performed neither the exact material nor its approximate strength properties were known. At that time it was assumed the material was normal structural grade steel; and because of the high strain rate associated with the test and the work hardening introduced in forming the corrugations, a yield strength of 60 ksi was used. Subsequently it was learned the material used was essentially commercially pure iron which has a static yield strength of 25-30 ksi in the corrugated state. The dynamic increase in yield strength accounts for the 40 ksi mentioned above.

The following discusses briefly the behavior of the corrugated arches which were tested in Operations Hardtack and Plumbbob. Because of their importance, the strength characteristics of these arches is summarized in detail here. The tension-shear-bearing (T:S:B) ratio for standard Armco details were computed. The results of these computations are given below.

Assumptions:

σ_y = 40 ksi under dynamic conditions for general yielding of plate

τ_y = 60 ksi under dynamic conditions on the shank of high strength bolts

τ_y = 45 ksi under dynamic conditions on the shank of standard structural bolts

σ_b = 170 ksi under dynamic conditions for bearing of bolts

(A 25% increase in static values was assumed uniformly above in inferring the dynamic yield strengths).

For 10 ga material (Used in Hardtack 3.2a, b, and c, and in Plumbbob 3.3b):

Allowable tension(or compression) = T = 40 x 2.003 = 80^k/ft

Allowable shear on bolts = S = 4 x 0.44 x 45 = 79.2^k/ft*

Allowable bearing on bolts = B = 4 x 0.75 x 0.135 x 170 = 69.0^k/ft.

T:S:B = 1:0.99:0.86

For 1 ga material (Used in Hardtack 3.2d only).

The report does not state the number of bolts used in the longitudinal joints for this structure. However, the pictures in the reference indicate a 6 bolt/ft. joint. These joints were bolted with high strength bolts.

Allowable tension(or compression) = T = 4.118 x 40 = 165^k/ft

Allowable shear on bolts = S = 6 x 0.44 x 60 = 158^k/ft

Allowable bearing on bolts = B = 6 x 0.75 x 0.276 x 170 = 211^k/ft.

T:S:B = 1:0.96:1.28

Thus the above values indicate that the joints are essentially balanced in design with regard to "tension" and shear. Also the 10 ga

*Computed for standard bolts only since standard bolts were used in the 10 ga Hardtack and Plumbbob structures.

structures are critical in bearing while the 1 ga structure is critical in shear or in general yielding of the plate. In the Hardtack tests the three 10 ga structures (3.2a, b, and c) all had evidence of bearing failures in the joints. One picture Ref. (F.2) shows the bolt holes severely elongated and apparently the bolts actually fell away from the joint. This last observation, however, probably occurred during the negative phase or during the excavation of the structure following the test. Failure of the 1 ga structure (3.2d) apparently was by general yielding of the plate, but the final shape of this structure was unusual in that it corresponds almost exactly to the configuration to be expected had the structure failed in its second mode by buckling. Normally a structure which has an average depth of cover of one-fourth the span would not be expected to fail by buckling since the deformations associated with the buckling would mobilize resistance in the soil which would prevent this buckling. It is interesting to note that the critical buckling loads of these arches under uniform external pressure without earth cover would be 5.6 psi and 3.4 psi for the 10 ga and 1 ga structures respectively in the second mode. The static overburden (based on average depth of burial) would produce static uniform loads of 5.2 and 6.1 psi respectively on the 10 ga and 1 ga arches. As already mentioned however this overburden, although sufficient to produce buckling if it were applied as a fluid pressure, prevents buckling by virtue of its ability to mobilize strength through deformation. Thus, even though the final deflected shape and the preceding values of load indicate that failure occurred by buckling, it is believed to be merely a curious coincidence. Instead it seems more reasonable on the basis of the following considerations to explain the observations pertaining to Structure 3.2d

on the basis of the initial deformations produced as a result of the backfilling which actually triggered the failure.

From Table 2.3 of Ref. (F.2), it may be shown that the crown of structure 3.2d was displaced upward by 9.2 in. while at the quarter points the arch was displaced inward by 4.3 in. on the "lee" side and by 4.9 in. on the "windward" side. These deformations are three or more times the similar deformations for the three lighter structures. These deformations indicate by their magnitude and their direction that failure could have been imminent under the static loads and the final failed configuration is a magnification of these initial displacements. Furthermore, it is difficult to conceive the shock loading causing the observed failure had these initial displacements not been present. This observation emphasizes the necessity of retaining the initial circular shape by interior shoring and/or preloading the crown of light gage structures during backfilling.

Using the methods given in Ref. (F.1) gives the estimated overpressures to cause failure of the Hardtack and Plumbbob structures shown in Table F.2.

A corollary of the great significance to the subject contract from the above tests and observations is that it is impossible to provide the desired overpressure protection in the size of structure required by use of unstiffened light gage corrugated metal arches. The desired protection can be obtained in stiffened corrugated arches, but the considerations of weight, ease of construction and versatility continue to indicate that the steel arch ribs supporting timber blocks are the most practical configuration for the structure required.

The computations summarized in Tables F.1 and F.2 illustrate that for the particular structures considered a moderate kiloton weapon produces a forcing function of practically an infinite duration on the structure. That is, the peak overpressure causing failure from a moderate kiloton weapon is practically the same as that from a large megaton weapon. Test experience with other types of construction indicates that this near equality of overpressure should not occur; for failure with a large megaton weapon the peak overpressure should be significantly less than the peak overpressure for failure from a moderate kiloton weapon. Therefore, it was desirable to obtain a procedure which was more dependent on weapon yield. The development of this procedure follows immediately.

THEORETICAL EFFECT OF FOOTING MOTION

It has been observed Ref. (F.3) that the rise time (the time required for the stress to build up from zero to its maximum value) of the vertical stress in soil increases as the depth below the ground surface increases. If it is assumed that this increase is linear with depth, the geometry shown in Fig. F.2 specifies for an ideal shock wave

$$k = \frac{t_d}{d}\left(1 - \frac{p}{p_m}\right) \tag{1}$$

In an unpublished study* performed by Capt. J. E. Maloney, CE, USA, while a student at the University of Illinois, it was shown that the duration of the triangular force pulse which reproduces the area under an ideal pressure-time curve as defined in Ref. (F.4) may be stated as

*A more recent study (Newmark and Hall, "Preliminary Design Methods for Underground Protective Structures, 1961 Revision," AFSWC TDR 62-6) gives $t_d \cong \dfrac{6.4\, W^{1/3}}{p_m^{0.6}}$ for p_m not less than 30 psi.

$$t_d \cong \frac{10\, W^{1/3}}{p_m^{\ 2/3}} \quad \text{with } p_m \text{ not less than 30 psi} \tag{2}$$

where t_d = duration of triangular force, sec.

W = weapon yield, MT

p_m = peak side-on overpressure, psi

Substituting (2) into (1) yields

$$k = \frac{10\, W^{1/3}}{d\, p_m^{\ 2/3}} \left(1 - \frac{p}{p_m}\right) \tag{3}$$

In Ref. (F.5) values of p/p_m have been evaluated as a function of depth for overpressures of 100 and 200 psi produced by hypothetical weapons of 40, 1000, and 5000 kt yield. The value of k is determined from these data in Table F.3 where it is seen that this quantity may be taken as

$$k = 18 \times 10^{-4} \text{ for d not in excess of 50 ft.} \tag{4}$$

It was found in making the computations in Table F.3 that k was very sensitive to small changes in p_m/p but that p_m/p was quite insensitive to large changes in k. Therefore, treating this parameter as constant is justified. However, the data in Ref. (F.5), as one would expect, indicate attenuation of vertical stress with depth in soil is not linear. An assumption of linearity seems reasonable though up to a depth of 50 ft. Therefore, Eq. (4) is restricted to depths of 0 to 50 ft.

Equating (3) and (4) yields

$$p/p_m = 1 - \frac{18 \times 10^{-5} p_m^{\ 2/3} d}{W^{1/3}} \quad \text{for d not in excess of 50 ft.} \tag{5}$$

This last equation expresses the attenuation of vertical stress in soil, but since the data in Ref. (F.5) were derived for a soil with a seismic velocity of 2000 ft./sec, Eq. (5) is probably only applicable for soils possessing approximately this seismic velocity. Since the practical range of this velocity for soil is approximately 1000 to 5000 ft./sec, however, Eq. (5) could represent a reasonable average for most soils.

Now the response of a single-degree-of-freedom system of the type shown in Fig. F.1 subjected simultaneously to an applied force and a base motion is expressed by the differential equation

$$m\ddot{u} + f(u) = p(t) - m\ddot{x} \tag{6}$$

where m = mass

u = relative displacement of mass and base

$p(t)$ = forcing function applied to mass

$\ddot{x}$ = absolute acceleration of base

Thus, the net force acting on such a system is expressed by the difference between the applied force and the inertia force resulting from the acceleration of the base.

Also in Ref. (F.5) it is shown that the acceleration of the soil is directly proportional to the slope of the pressure-time curve, or

$$\ddot{x} = 0.533 \frac{\Delta p}{\Delta t} \tag{7}$$

where $\ddot{x}$ = acceleration, in./sec^2

Δp = change in pressure, psi

Δt = change in time, sec.

Now if we let d_1 and d_2 be two different depths below the ground surface in feet, Fig. F.5 may be constructed from which the values of the positive $(\dot{x}_+)$

and negative ($\ddot{x}_-$) accelerations at a depth d_2 are computed. Thus in Fig. F.3 the curved transitions at the value of peak pressure are assumed to avoid a jump discontinuity in the accelerations. The net pressure curve acting at a depth d_1 resulting from the above derivation is shown in Fig. F.4 wherein the significance of p and p' are noted.

$$\ddot{x}_+ \cong 300 \frac{p_m}{d_2}\left[1 - \frac{18 \times 10^{-5} p_m^{2/3} d_2}{W^{1/3}}\right] \tag{8}$$

and

$$\ddot{x}_- \cong \frac{1}{19} \frac{p_m^{5/3}}{W^{1/3}}$$

From Eq. (6) and by observing Fig. F.4 it may be stated

$$p \cong p_2 + m\ddot{x}_-$$

or

$$p/p_m = 1 - \frac{18 \times 10^{-5} p_m^{2/3}}{W^{1/3}} (d_2 - 300m) \tag{9}$$

But if the mass of an arch rib is small in comparison to the mass of the soil overburden which it generally is for structures meeting the criterion for complete burial,

$$m \cong 180 \times 10^{-5}\ d_1 \tag{10}$$

where m = mass in lb-sec^2 - in.$^{-3}$

d_1 = average depth of overburden, ft.

Therefore:

$$p/p_m = 1 - \frac{18 \times 10^{-5} p_m^{2/3}}{W^{1/3}} \left(d_2 - \frac{d_1}{2}\right) \tag{9'}$$

It should be noted that Eq. (9) can yield a zero or even a negative value of p for certain combinations of the variables. When this occurs,

p' may become the maximum net pressure instead of p; from Eq. 6 as illustrated in Fig. F.4:

$$p' = p_1 - m\ddot{x}_+$$

$$p'/p_m = 1 - \frac{300m}{d_2} + \frac{18 \times 10^{-5} p_m^{2/3}}{W^{1/3}} (300m - d_1) \qquad (11)$$

Or neglecting the mass of the arch rib

$$p'/p_m = 1 - \frac{d_1}{2d_2} - \frac{9 \times 10^{-5} p_m^{2/3} d_1}{W^{1/3}} \qquad (11')$$

A criterion could be established to define the range of applicability of Eq. (9') or (11'). However, it seems simpler to compute the magnitude of both p and p' and to use the maximum value of the two.

COMPARISON OF THEORETICAL EFFECT OF FOOTING MOTION AND MEASUREMENTS IN SOIL

The procedure derived above cannot be compared directly with the results of the tests of arches and conduits in Operation Plumbbob since in most instances the accelerations of the footings were not measured directly. Yet, the measurements in the soil where there was no structure (i.e., the free medium) provide a means of empirically checking the above procedure. In Operation Plumbbob both Sandia Corporation (Ref. F.6) and Stanford Research Institute (Ref. F.7) measured free medium accelerations at several depths below the ground surface at ranges corresponding to specified values of surface overpressure. From these data the net pressure acting in the free medium at some depth d_1 below the ground surface taking account of the motion at some greater depth d_2 below the surface consistent with the values derived in the preceding section could be computed. The depth below the ground surface chosen for computing the net pressure was five feet which

corresponds approximately to the depth of burial of the arches and conduits tested in Operation Plumbbob. The depth d_2 was chosen to correspond to the depth at which acceleration measurements were taken.

The net pressure computation was carried out by:

(a) Determining the attenuation of vertical stress in a depth of 5 ft. from Ref. (F.5).

(b) Constructing a curve for vertical stress at a depth of 5 ft. by modifying the ordinates of the measured surface overpressure by the amount of attenuation from (a). At the same time all coordinates of the surface overpressure were advanced by 5 msec. to account for the observed seismic velocity in the surface layers of Frenchman's Flat of 1000 ft./sec.

(c) Constructing a curve showing the variation with time of the product of the mass of soil to a depth of 5 ft. and the acceleration at the depth where the acceleration was measured.

(d) Subtracting algebraically the curve discussed in (c) from the curve discussed in (b) to find the net pressure variation with time.

Since attenuation with depth is defined in Ref. (F.5) for only the 100 and 200 psi overpressure levels only these overpressures were considered in this net pressure computation. In Fig. F.5 the net pressures and the measured surface overpressures are compared for these two cases. It will be noted that the net pressures are 80 and 50 percent of the measured overpressure for 100 and 200 psi respectively. These percentages compare

favorably with the maximum values computed from Eq. (9') and (11'); namely, 86 percent for both pressure levels considered. Note the difference in d_2 for the two cases stated in Fig. F.5. The theoretical value is decidedly high in the second case; yet its use is obviously conservative. Figure F.5 illustrates that the shape of the net pressure curve may be decidedly different from the shape of the surface overpressure curve. However, these two comparisons indicate the effective durations of the net pressure may be somewhat less than the comparable duration of the overpressure; thus use of the effective duration of the overpressure in conjunction with the net pressure would appear to be a conservative assumption. This assumption is made in the following computations which compare the results of the tests of the arches and conduits in Operation Plumbbob with strength predictions based upon the method developed earlier in this appendix.

COMPARISON OF THEORETICAL EFFECT OF FOOTING MOTION AND OBSERVED BEHAVIOR OF ARCHES

The preceding derivation considers only the behavior of the soil with no structure present. However, if perfect coupling between the soil and a structure buried in the soil is assumed, the procedure developed may be applied to determine the net forces on the structure. In this section a comparison of the results obtained from this assumption is presented.

Nineteen arch and conduit sections were tested in Shot Priscilla of Operation Plumbbob. Seven of these were semi-circular arches of which four were fabricated from reinforced concrete (Ref. F.8) and three were fabricated from corrugated metal (Ref. F.9). On the latter three, two contained rolled sections which formed circumferential stiffeners. The remaining twelve sections tested (Ref. F.10) were divided among three

circular reinforced concrete sections, two circular corrugated metal sections, and seven corrugated metal "cattle pass" sections. All of these structures met the criterion of full burial specified in Ref. (F.1).

The dynamic resistance and natural period of vibration of these sections computed on the assumption of hoop compression is summarized in Table F.4. For the corrugated metal sections two modes of failure, general yielding of the plate material and shearing of the bolts across their shank diameter, were considered. Only these modes of failure were considered as a result of the conclusion that buckling of buried structures of practical proportions is highly improbable and that bearing failure of bolted joints acting in compression can hardly cause primary failure of the joint.

There is reason to believe that failure of steel in shear would exhibit less ductility than failure under direct stress or bearing. As a result failure of such structures would probably be governed by shearing of the bolts under dynamic conditions provided that the amount of friction developed in the joint is sufficiently small to allow the bolts to go into shear.

If buckling of the reinforced concrete sections is impossible, and from the observed behavior it certainly appears it is, these sections could fail only by crushing of the concrete under the action of the direct stress. The computed resistance and natural period of vibration for these sections in Table F.4, therefore, are based on this mode of failure.

By use of the dynamic strengths summarized in Table F.4, the overpressure levels necessary to cause failure of the arches and conduits tested in Operation Plumbbob were computed using the net pressures defined

earlier. In the original computations the strength of the joints with structural bolts was assumed to occur at a shearing stress of 30 ksi. However, the actual strength based upon the test results (Ref. F.11) appears to be 45 ksi. As a result the overpressures from the original computations were increased by the ratio of these stresses. For the same reason given earlier this correction is not rigorous but the error introduced in this case is relatively insignificant. The overpressure levels computed on this basis are compared in Table F.1 with the observed levels from the test and with the levels computed from assuming the foundations immovable. It is apparent in this comparison that the significance of the effect of footing motion increases as the damage pressure level increases.

The computation of overpressure level causing failure in buried arches incorporating the effect of footing motion was carried out in the following manner:

(a) To obtain an estimate of the effective duration of the loading function an overpressure level is assumed.

(b) The maximum value of the net pressure (p) acting at the average depth of burial of the arch is computed from Eq. (9') or (11'). (For the "cattle passes" and circular conduits the average depth of burial was assumed to be the same as for a semi-circular arch with the same radius as the upper portion of the conduit.)

(c) Estimate the effective duration (t_e) consistent with the assumed overpressure level.

(d) Compute the ratio of the effective duration to the natural period of vibration of the structure.

(e) Assume a value of μ (the ratio of failure deflection to the yield deflection.) For reinforced concrete sections μ was taken to be 5. Five was also used for the metal structures failing by shearing of the bolts. For metal structures failing by general yielding a value of 20 was assumed.

(f) From Fig. 7 of Ref. (F.1), the values of the ratio of time of maximum response (t_m) to the natural period of vibration and the ratio of the maximum net pressure (p) to the yield resistance (r) of the structure were obtained.

(g) Compute t_m and compare it to t_e. If the assumed t_e approximately reproduces the area under the overpressure curve up to t_m continue to (h); if it does not, re-estimate t_e and proceed from (d).

(h) Compute p and compare it to the value of p computed in (b). If the two values are the same the assumed overpressure is the desired answer; if they are not the same, a new overpressure is assumed and all computations repeated.

DISCUSSION AND CONCLUSIONS

A method for accounting for the effect of footing motion upon the response of buried structures was developed which correlates reasonably well with measurements in the free medium. It also estimates a damage pressure level which appears consistent with the observed behavior of the fully buried arches and conduits which were tested in Operation Plumbbob. Somewhat greater confidence in this method would be established had some of the test structures actually collapsed. However, the derivation of the expressions related to the net pressures is entirely rational

(The attenuation study in Ref. (F.5) also is entirely rational.) so that the correlations noted seem to validate the procedure at least within the range of the observations. Yet a serious question exists regarding extrapolation of the results of this study. It already has been noted that the linearity of the dependence of rise time upon depth must be limited to depths of approximately 50 ft. Since the theoretical study of the attenuation of vertical stress with depth was limited to overpressures of 100 and 200 psi, application of the method outside of this range was inferred on the basis of the theory. The attenuation study assumed the soil remained elastic under the dynamic loading and the computed attenuation was the result of three dimensional spreading and of the non-uniformity of the distribution of the loading over the ground surface.

Some permanent deformation of the ground surface was observed in Shot Priscilla at both the 100 and 200 psi pressure levels, but this deformation was relatively minor. There is a small amount of data on the subject, but the existing data indicate major permanent deformation of the ground surface in Nevada soil is limited to the range corresponding approximately to the maximum radius of the fireball. This observation might suggest an upper limit to the overpressure for which the method might be applicable of perhaps 1000 psi, but it should be emphasized that the data justifying this limit are very meager.

The non-uniformity of distribution of overpressure over the ground surface of course exists for all magnitudes of pressure. Yet as the level increases the non-uniformity of the initial phase of the pressure becomes decidedly more pronounced. The effect of the initial decay of the overpressure is not at all obvious from the quantitative standpoint although

it is apparent that the attenuation must decrease with decreasing pressure. Study of the development of the theory given herein indicates the procedure does specify a decreased attenuation as the overpressure decreases. This results from the fact that the increase in rise time is a linear function of depth alone. Therefore, at the same depth at ranges corresponding to two different overpressure levels, the rise curve would intersect the decay curve at a higher percentage of the peak side-on overpressure for the smaller of the two overpressures because of the lesser rate of decay of the smaller overpressure. This phenomenon is illustrated in Fig. F.6 by noting additionally that the actual positive phase duration is practically constant for overpressures in excess of approximately 30 psi. Observation of this implicit behavior of the theory developed herein suggests that this theory may be applicable down to overpressure levels of 30 psi. The limitation of Eq. (2) specifies the magnitude of 30 psi as the lower bound on overpressure.

In summary therefore it may be stated that Eq. (9') and (11') defining the net pressure acting on a fully buried structure are applicable for depths below the ground surface which are less than 50 ft. and for overpressures ranging from 30 to 1000 psi. Also the effects of footing motion upon the net forces acting on a structure decrease as the size of weapon increases. For weapons of approximately operational size this effect becomes practically insignificant.

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TABLE F.1 COMPARISON OF DAMAGE PRESSURE LEVELS WITH OBSERVED EXPERIENCE IN OPERATION PLUMBBOB

Structure No. and Description	Weapon Yield	Overpressure for Failure by General Yielding: Footings Immovable	Overpressure for Failure by General Yielding: Footings Movable	Max. Observed Overpressure	Overpressure for Failure by Shearing of Bolts: Footings Movable
---		psi	psi	psi	psi
3.1a, b, c, and n---Buried R/C Semi-Circular Arch---8'-0" radius	Moderate kiloton	480	600	190	---
	Large megaton	350	350	---	---
3.2e, j, and l Buried R/C Pipe 8'-0" diameter	Moderate kiloton	750	1100[b]	136	---
	Large megaton	580	550	---	---
3.2d and h---Buried 10 ga. Corrugated Metal Pipe---8'-0" diam.	Moderate kiloton	190[a]	200	136	165[a]
	Large megaton	170[a]	150	---	130[a]
3.2a, b, c, f, g, k, and m--- Buried 10 ga. Corrugated Metal "Cattle Passes" ---2'-6" radius	Moderate kiloton	330[a]	400	153	450
	Large megaton	250[a]	250	---	275
3.3b---Buried 10 ga. Corrugated Metal Semi-Circular Arch Without Stiffeners---12'-6" radius	Moderate kiloton	70[a]	75	56	60[a]
	Large megaton	50[a]	50	---	45[a]

[a]Corrected for yield strength but not for effective duration. Correcting the effective duration would result in a somewhat lower value for the moderate kiloton case.

[b]A result possibly outside the range of applicability of the theoretical expression.

TABLE F.2 COMPARISON OF DAMAGE PRESSURE LEVELS WITH OBSERVED OVERPRESSURE IN OPERATIONS HARDTACK AND PLUMBBOB

Structure No.	Overpressure to Cause $\mu = 5$ (Footings Immovable)	Overpressure to Cause $\mu = 20$ (Footings Immovable)	Observed Overpressure	Weapon Yield
Operation Hardtack:				
3.2a	60 psi	75 psi	90 psi	Low kiloton
3.2b	40 psi	50 psi	78 psi	Low megaton
3.2c	40 psi	50 psi	180 psi	Low megaton
3.2d	60 psi	70 psi	100 psi	Low megaton
Operation Plumbbob:				
3.3b	50 psi	65 psi	56 psi	Moderate kiloton

TABLE F.3 DETERMINATION OF THE PARAMETER k FROM EQUATION (3) AND FROM DATA IN REF. (4)

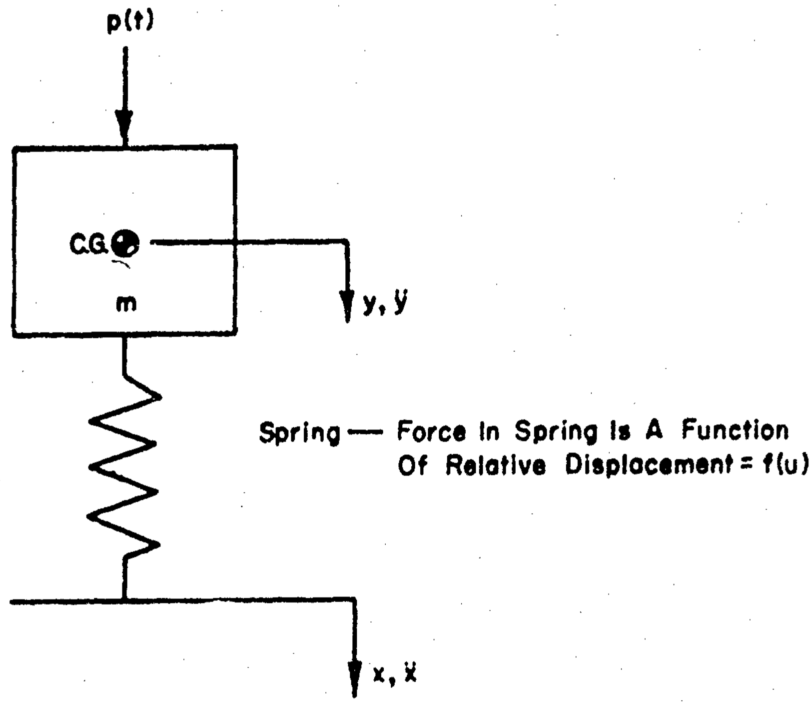
Weapon Yield	Depth	Peak Side-on Overpressure	Ratio, Peak Stress at Depth (d) to Peak Overpressure	k
kt	ft.	psi	---	$sec\text{-}ft^{-1}$
40	5	100	0.95	0.00159
40	10	100	0.90	0.00158
40	15	100	0.85	0.00159
40	50	100	0.63	0.00117
40	5	200	0.90	0.00200
40	10	200	0.82	0.00180
40	15	200	0.75	0.00168
1000	50	100	0.85	0.00139
5000	50	100	0.88	0.00159

TABLE F.4 DYNAMIC PROPERTIES OF BURIED ARCHES AND CONDUITS TESTED IN OPERATION PLUMBBOB

Structure Number	Ave. Depth of Burial	Depth to Footing	Yield Resistance General Yielding	Yield Resistance Bolt Shear	Natural Period of Vibration
---	ft.	ft.	psi	psi	msec.
3.1a	5.7	12.7	360[a]	---	11.5
3.1b	5.7	12.7	360[a]	---	11.5
3.1c	5.7	12.7	360[a]	---	11.5
3.1n	5.7	12.7	360[a]	---	11.5
3.2e	8.4	16.3	550[b]	---	6.4
3.2j	8.4	16.3	550[b]	---	6.4
3.2l	8.4	16.3	550[b]	---	6.4
3.2d	8.4	15.5	130	130	16.0
3.2h	8.4	15.5	130	130	16.0
3.2a	8.0	15.2	220	275	9.0
3.2b	10.5	17.7	220	275	10.8
3.2c	8.0	15.2	220	275	9.0
3.2f	5.5	12.7	220	275	7.8
3.2g	8.0	15.2	220	275	9.0
3.2k	8.0	15.2	220	275	9.0
3.2m	5.5	12.7	220	275	7.8
3.3b	7.7	17.5	45	45	50.0

[a]Resistance defined by crushing of concrete for f'_c = 4300 psi.

[b]Resistance defined by crushing of concrete for f'_c = 3000 psi.



Differential Eq. Of Motion:

$m\ddot{u} + f(u) = p(t) = m\ddot{x}$
m = supported mass
p(t) = force applied to mass
u = y − x = relative displacement of mass and base
x = displacement of base
$\ddot{x}$ = acceleration of base
y = displacement of mass
$\ddot{y}$ = acceleration of mass

FIG. F.1 MATHEMATICAL MODEL AND NOTATION USED IN EQUATION 6.

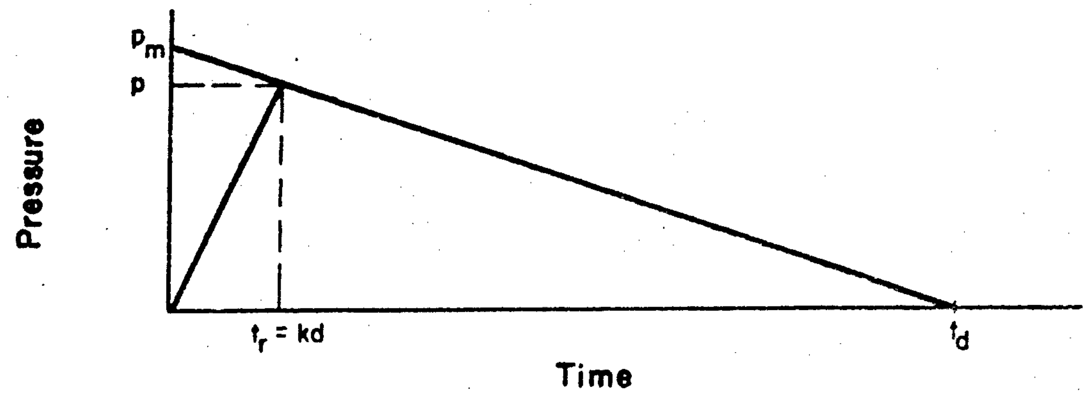


FIG. F.2 GEOMETRY OF IDEAL SHOCK AND IDEAL VERTICAL STRESS IN SOIL.

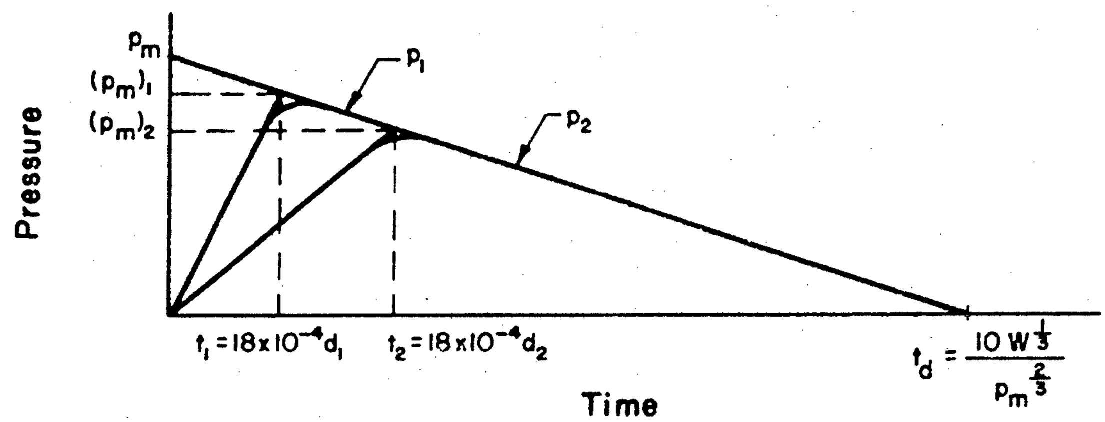


FIG. F.3 IDEAL VERTICAL STRESS AT TWO DEPTHS IN SOIL FOR AN INFINITE SEISMIC VELOCITY $(d_2 > d_1)$

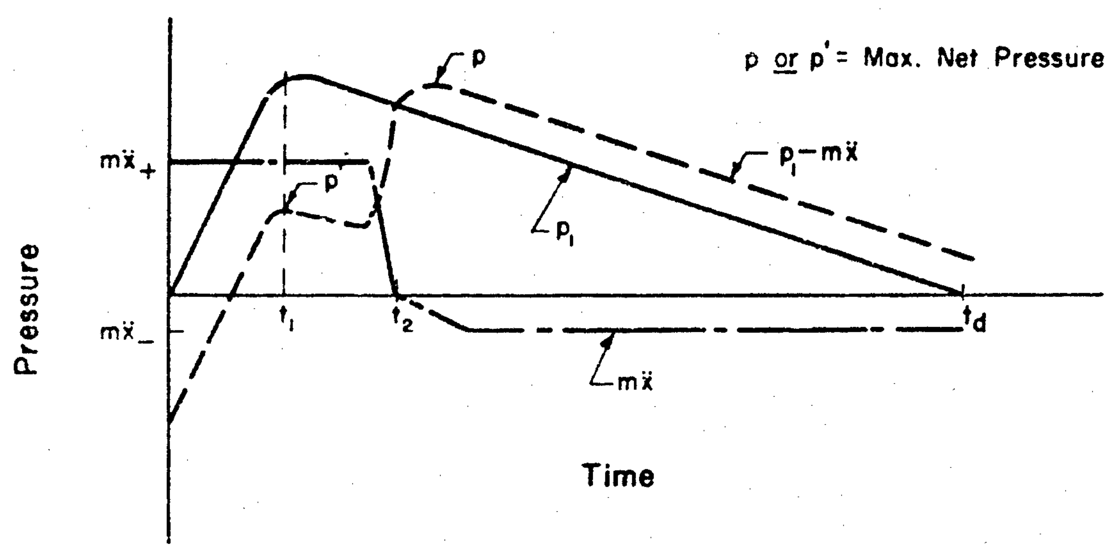
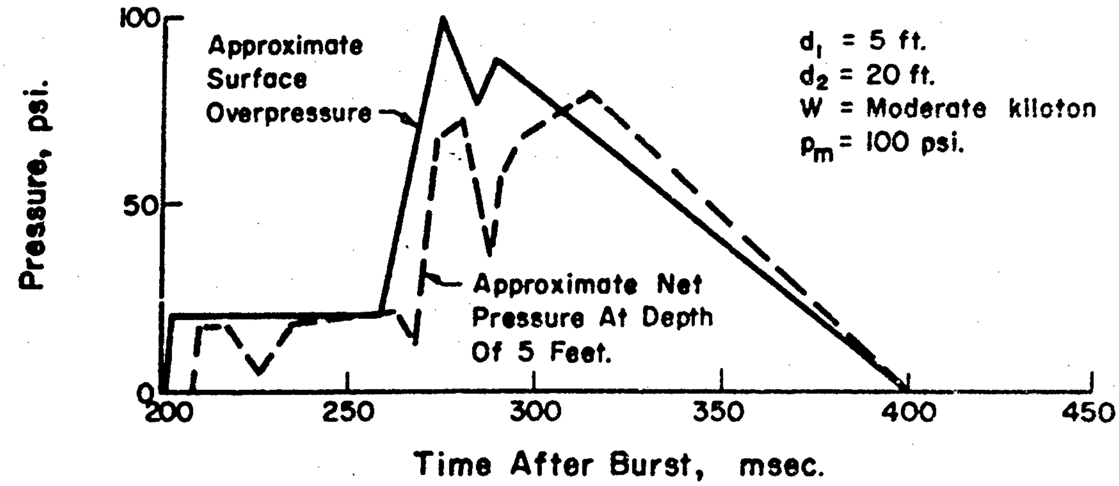
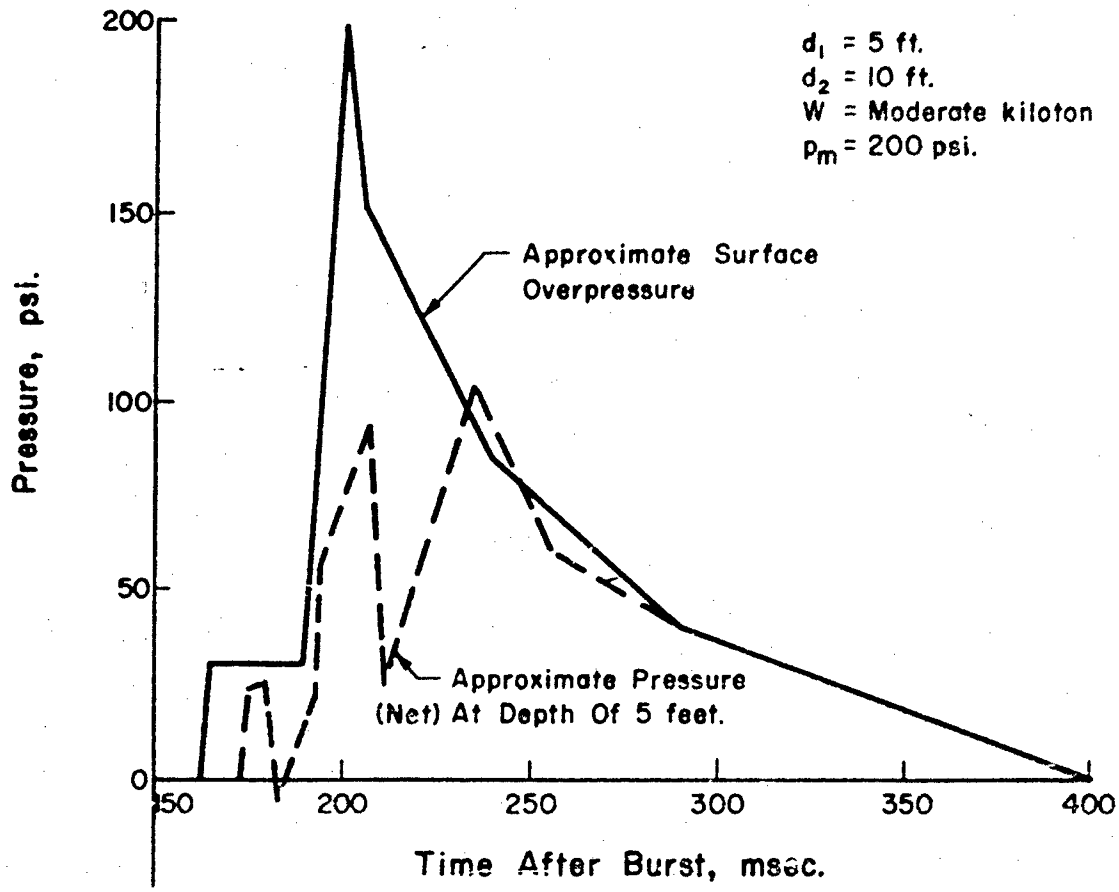


FIG. F.4 IDEAL NET PRESSURE DERIVED FROM FIGURE F.3



(a) 100 psi. Overpressure



(b) 200 psi. Overpressure

FIG. F.5 NET PRESSURE IN FREE MEDIUM FROM MEASUREMENTS MADE IN OPERATION PLUMBBOB.

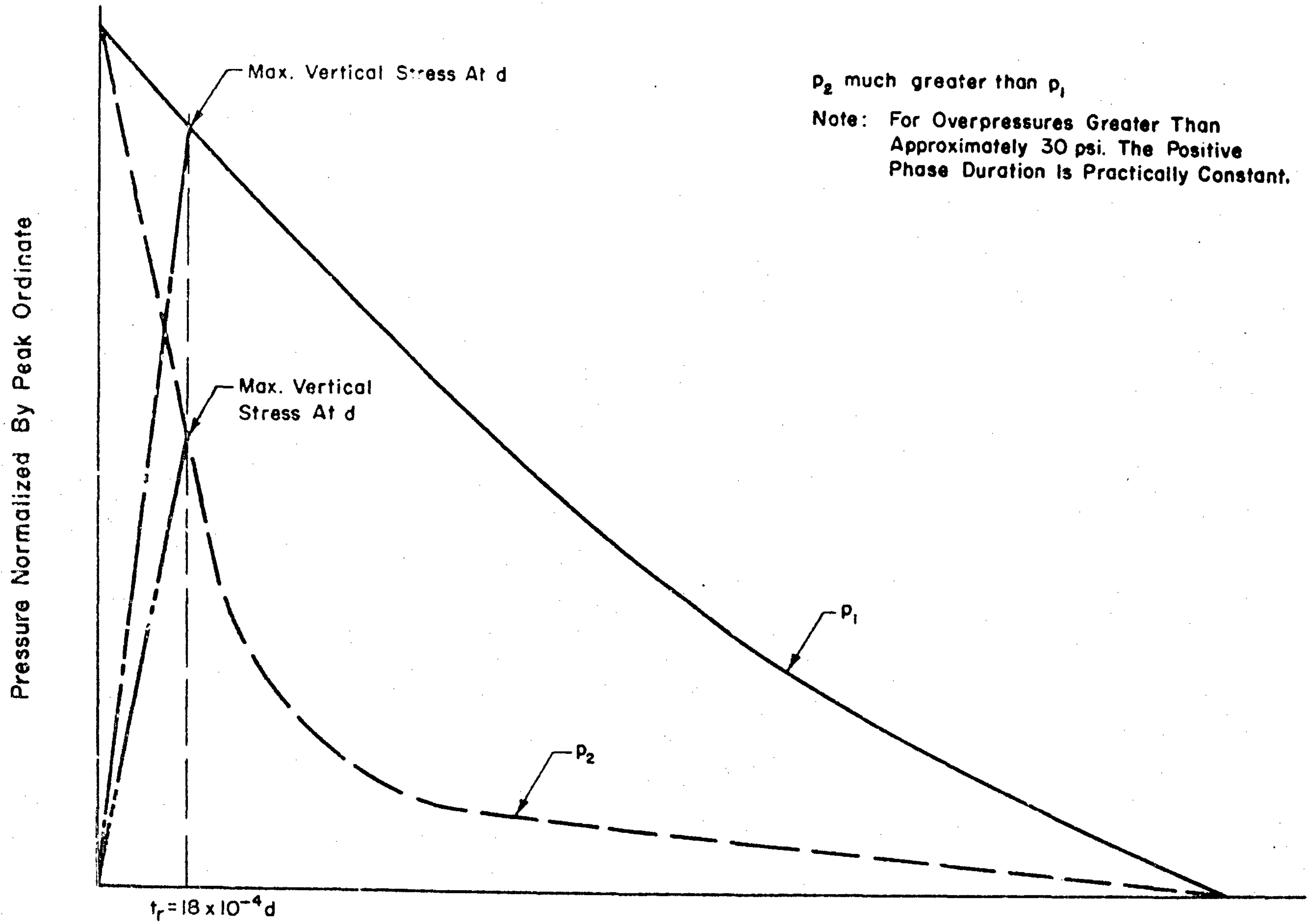


FIG. F.6 EFFECT OF OVERPRESSURE LEVEL ON ATTENUATION OF VERTICAL STRESS IN SOIL.

APPENDIX G

SPACING BETWEEN BURIED STRUCTURES

INTRODUCTION

Where a number of structures are to be placed in the same general area, it becomes necessary to consider the spacing required between them in order that the structural behavior under dynamic conditions, considered in the design of the structures, can be mobilized and in order to insure that the resistance of the soil required to develop the strength of the structures will be available. In general, no serious problems exist insofar as arch structures are concerned placed end to end in a single line. The forces on the end bulkheads of adjacent structures are not seriously effected by the placing of an adjacent structure. However, the situation is not so simple when arch structures are placed side by side. The reason for this is that the behavior of the arch, supported by the soil, involves thrusts and compressions in the soil over the arch which must be transmitted to the adjacent soil at or above the level of the footings. If an adjacent arch barrel intersects the line of thrust mobilized in the soil, this implies greater radial loads on the adjacent arches, with consequent greater tendencies to deflect or to fail.

It is the purpose of this Appendix to explain the basis for the recommendations regarding spacing between buried flexible structures. Unfortunately, adequate data for a rigorous definition of this problem are not available, either from field test or from analysis. Hence, the development presented herein is heuristic, and the recommendations are based primarily on judgment and experience.

MODE OF ACTION OF BURIED FLEXIBLE STRUCTURES

For a flexible structure buried below the surface of a mound or below the horizontal surface of the soil, the loading on the structure consists primarily of a nearly uniform vertical compressive force, varying with time, with corresponding lateral or horizontal forces of a somewhat smaller magnitude, depending on the characteristics of the soil cover. There is a small component, lasting briefly, of an unbalanced vertical force, greater on one side than on the other, which tends to produce an unsymmetrical or deflection type of response. In the case of a structure under a mound, because of the longer duration dynamic forces acting on the mound, the unsymmetrical loading on the structure may last for a longer time.

However, in either case, the deflection of the structure consists essentially of a major and a minor component. The major component is a compression more or less uniform around the structure, which tends to shorten the arch axis, and which tends also to push the footings down into the soil. The compression mode of deformation is symmetrical. The stresses in the structure must not exceed those causing a general yielding of the entire arch in this mode, or else the structure will be able to deform unsymmetrically, or buckle, by permitting one or the other half of the arch to push through into the opening.

The second, and generally minor, mode of deformation involves a lateral displacement of the arch, bulging inward on one side and outward on the other. This arises from unsymmetrical loading by the forces enveloping the structure. However, even though the loading might tend to be unsymmetrical, the bulging outward of the arch tends to push the soil up and out also, and generates a passive resistance in the soil which tends

to reduce the unsymmetrical component of deformation and which makes the resultant loading more nearly symmetrical. Hence, when the structure is properly backfilled, and where there is a sufficient depth of cover over it, lateral deformations and deflections are inhibited and the net deformation is primarily compression.

The amount of cover that is sufficient to cause the structure to deform nearly uniformly is a matter that depends on the type of material, the way it is treated in the backfill, and the configuration of the fill or mound over the structure. However, on the basis of field test data and approximate analysis, an amount of cover of the order of 1/8 of the span over the crown of the arch will generally be sufficient to mobilize the resistance of the soil and to prevent any large unsymmetrical deformation. Under certain circumstances, even a smaller depth of cover will be sufficient, but only rarely unless the cover is very poor and very poorly placed will additional cover be required beyond this amount in order to inhibit the lateral deformation. The mere mass of the material that has to be moved, with the amount of cover specified above, will tend to prevent or inhibit dynamic lateral deformation even though it may not be sufficient to inhibit static lateral deformation. Nevertheless, extensive void spaces or very soft material on one side of the arch as compared with the other would emphasize the tendency to lateral movement and would provide an unsafe condition.

It may be concluded that where the amount of cover is sufficient to cut down the magnitude of the lateral deformation of the flexible arch, one need not be concerned with the moments in the arch, as computed for any system of loads applied to it, because the loading is changed by the

resistance of the soil to a loading that will correspond to the shape of the arch itself, and the magnitude of the moments will be reduced accordingly. Consequently, under the conditions provided herein, the forces for which the structure must be designed involve primarily the compressive dynamic load of the air blast, transmitted through the soil to the arch, considered as essentially uniform on the entire arch, although varying with time, and the resistance provided by the arch must be essentially equal to its yield strength under dynamic loading.

SPACING BETWEEN BURIED STRUCTURES

The spacing between adjacent buried structures, then, should be sufficient to prevent unbalanced lateral loading or reduced lateral resistance in the soil. When structures are spaced very close together laterally, the amount of soil that must be moved in order to provide dynamic lateral resistance is reduced, and the resistance is not inhibited nearly so much as in the case where the structures are based far apart. Moreover, the compressive load carried by the soil around the flexible arch tends to react against the arched surface of the adjacent arch, and produces a higher intensity of loading on the adjacent arch, on one side.

The problem is essentially not unlike the problem of stress concentration in a plate with a row of holes in it. Under uni-directional loading in such a plate, the stress concentrations are not changed materially from those for a single hole in the plate if the spacing between the holes is equal to or greater than their diameter.

Based on considerations of the stress distribution around holes in general, and considering the disturbance of the soil arising from dynamic loads transmitted through the soil when a structure is embedded in it, it

is likely that the area within which major disturbances lie, around a structure embedded in soil, extends outward from the structure about half the diameter of the structure on both sides. However, the disturbance tends to decrease as the distance increases, and hence the major effect of the interruption of the continuity of the soil by the structure occurs within a distance of the order of one-half the radius on each side.

From this point of view it appears that adjacent structures should be separated by a distance equal to at least one-half the sum of the radii of the adjacent structures, and preferably one-half the sum of the diameter of the adjacent structures. For structures of the same dimensions, the spacing between the structures, then, would be one structural diameter.

Practical considerations of backfilling the structure also suggest that the spacing between should be at least one diameter, so as to permit more effective backfilling if mechanical equipment is used, and to insure a sufficiently dense and strong backfill, in any event.

Moreover, the loads transmitted to the structure introduce fairly large forces on the footings, although these are of a transient nature. There can be a tendency for these footing forces to produce larger movements and settlements of the footings. One should avoid having forces transmitted from an adjacent structure to the footings, in addition to those transmitted from the structure of which the footings are a part. This is accomplished by having a spacing of the order of the average diameter of the adjacent structures, between adjacent structures.

In short, with a spacing equal to the average diameter of the two adjacent structures, the loading and the response of each structure is virtually independent of the adjacent structure. Hence this is the preferable

spacing if it can be achieved. Where conditions require a closer spacing, one should under no circumstances have a spacing less than half the above amount between adjacent structures. It must be considered, however, that if the spacing is reduced to the smaller amount, there may be some compromise in the strength and safety of the structure.

APPENDIX H

MODIFICATIONS FOR GREATER HARDNESS

One of the principal advantages inherent in the basic structure developed under this study results from the ease in which the structure can be modified directly in the field to provide a hardness in excess of that considered in the original design. It is the purpose of this appendix to indicate both the degree of hardness obtainable and to suggest means for obtaining this hardness. As an adjunct to the following discussion it will become apparent that the main structure has a resistance greater than that required to resist an overpressure of 100 psi; however, the amount of the excess resistance cannot be determined with precision.

BASIC RESISTANCE AND MODIFICATION OF MAIN STRUCTURE

In the original design of the main structure, the resistance to hoop compression was assumed to be mobilized only in the steel arch ribs, i.e., the timber blocks were assumed to act only in flexure spanning longitudinally between the ribs. To develop the concepts needed below, it is desirable here to re-evaluate the resistance in hoop compression of both the arch ribs and the wood blocks.

The ribs are structural tees (ST 4 WF 29) with a cross sectional area of 8.53 in.^{2}. If a static load r_s of 10 psi caused by the overburden and the weight of the structure is assumed, the stress induced in each rib with a spacing of 36.0 in. between centers of ribs would be, using notation from Appendix B.

$$\sigma_{sr} = \frac{r_s \, r'b}{A} = \frac{10 \times 96 \times 36}{8.53} = 4.0 \text{ ksi}$$

Therefore, for an effective dynamic yield point of 50 ksi, the reserve stress available to resist a dynamic load would be 46 ksi. For a ductility ratio μ of 5 and a blast load of infinite duration, the resistance of the arch rib to dynamic load would be

$$r_{DR} = \frac{\sigma_D A \left(1 - \frac{1}{2\mu}\right)}{r'b} = \frac{36 \times 8.53 \left(1 - \frac{1}{10}\right)}{96 \times 36} = 100 \text{ psi}$$

Similarly the 6" by 10" timber blocks spanning between the ribs have a flexural resistance for dynamic load (for $\mu = 2$) of:

$$\sigma_{SD} = \frac{Mc}{I} = \frac{10 \times 36^2 \times 6}{8 \times 5.5^2} = 0.32 \text{ ksi (stress from static load)}$$

$$r_{DB} = \frac{8\ \sigma_D\ I}{cL^2}\left(1 - \frac{1}{2\mu}\right) = \frac{8(7 - 0.3)\ 5.5^2}{6 \times 36^2}\left(1 - \frac{1}{4}\right) = 160 \text{ psi}$$

The horizontal shear in stress for the total load of 110 psi (100 psi blast load as limited by the ribs and 10 psi dead load), neglecting the dynamic effect is

$$v = \frac{3}{2}\frac{V}{A} = \frac{3 \times 110 \times 36}{2 \times 2 \times 5.5} = 540 \text{ psi}$$

Including the dynamic effect increases v to approximately 700 psi. These shear stresses are barely large enough to cause failure of the wood blocks under the conditions of loading specified. As pointed out in the original design calculations (Appendix C), the design of the wood blocks is controlled by horizontal shear and not by flexure. Thus, a resistance only for 100 psi blast load is provided.

The resistance of the wood blocks in hoop compression for allowable stress in compression parallel or perpendicular to the grain of 5 ksi and $\mu = 2$ may be computed as indicated. (Note the wood blocks are 35-1/4" long instead of 36" long to clear the stems of the structural tees.)

$$\sigma_s' = \frac{r_s\ br'}{A} = \frac{10 \times 36 \times 96}{5.5 \times 35.25} = 0.18 \text{ ksi (stress from static load)}$$

$$r_{DB}' = \frac{\sigma_D'\ A}{br'}\left(1 - \frac{1}{2\mu}\right) = \frac{(5-0.2) \times 5.5 \times 35.25}{36 \times 96}\left(1 - \frac{1}{4}\right) = 200 \text{ psi}$$

The relative stiffnesses of the arch rib and the timber in hoop compression would be with subscripts T and S referring to timber and steel respectively:

$$\frac{A_T\ E_T}{A_S\ E_S} = \frac{5.5 \times 35.25 \times 1.5 \times 10^3}{8.53 \times 30 \times 10^3} = 1.1$$

Thus, the stiffnesses are nearly balanced for the unmodified structure, and it might seem likely that the resistances in the two materials would initially develop with each material sharing approximately one-half of the load. However, in the immediately preceding calculations it is assumed that the timber blocks all develop the complete bearing area between adjacent blocks which is unlikely. It is probable that in the early stages of loading at least, the steel arch is much stiffer than the timber and, therefore it assumes most of the load. Consequently, although the standard structure would at first appear to be adequate in resisting a blast load of perhaps 300 psi (100 psi in the arch rib and 200 psi in the timber blocks in hoop compression - with the hoop compression reducing the reaction of the timber blocks on the ribs), one can say only that the hardness of the standard structure is greater than 100 psi but less than 300 psi, the actual hardness depending upon the quality of the fit between the timber blocks. In the general case, it is believed that the resistance of the standard structure might be consistent with a hardness of 150 psi, but until it is proved it would be dangerous to count on a hardness in excess of the 100 psi rating assigned to it in the main report.

On the other hand the preceding calculations suggest a simple field modification which will increase the hardness of the main structure to at least 200 psi and probably to more than 300 psi. Yet as just mentioned, the upper value should not be counted upon until it is proved. The simple field modification could be effected by ordering twice the number of arch ribs in each main structure kit (Kit 2) <u>but the same number of timber blocks and sills as in the basic structure</u>. At the outset it should be mentioned that other modifications would be equally sound structurally, but the one about to be described seems more economical of material and labor.

With twice as many structural tees they can be spaced at 1'-6" instead of the 3'-0" on centers by merely drilling additional holes in the sills and in the arch crown. Since these holes would be in timber they could be easily drilled in the field. The standard arch blocks (the timbers spanning between the steel ribs) could be cut to fit between the arch ribs. Cutting of the arch blocks would involve much labor in the field, but the 51 men doubtless could accomplish this modification rather quickly, particularly if a power saw is available. By reducing the spacing between arch ribs to one half their original value the resistances to dynamic load are increased as shown in the following table:

Member	Resistance to Blast Load of Infinite Duration psi	
Steel Arch Rib	210	
Timber Blocks:		
In Flexure	650	
In Horizontal Shear	210	(700 psi stress in timber)
In Hoop Compression	200	
Relative Stiffness:		
Timber-Steel (Hoop Comp.)	0.6	

On the basis of the earlier argument, the steel rib being stiffer than the timber will carry a greater portion of the load in hoop compression until it yields, when it will transfer additional load to the timber. The resistance of the steel rib alone is consistent with a hardness of 210 psi while the combined resistance in hoop compression of the rib and timber blocks is 450 psi assuming full contact between adjacent timber blocks and that the hoop compression reduces the reaction of the timber blocks on the rib to be consistent with 210 psi blast load. Inspection of the conditions in the outstanding leg of the structural tee indicates it is capable of resisting the reaction consistent with a 210 psi blast load (cf Appendix C). Also inspection of the footings indicates that the sills will not be overstressed even for a blast load of 450 psi (cf Appendix C) although the bearing stress would be significantly increased. Nevertheless, as pointed out in Appendix C, the bearing stress would not appear to be critical in limiting the hardness of the structure. Thus, it would appear that the fairly simple field modification outlined in the preceding discussion would increase the hardness of the main structure component to at least 210 psi and possibly to 450 psi. As already mentioned, it is believed that the modified structure might have a hardness of 300 psi.

MODIFICATION OF BULKHEADS AND APPURTENANT STRUCTURES

Increasing the resistance of the end bulkheads, passageway, utility structure, entranceway and ventilation structures is not as readily accomplished as is that of the main structure. Ease of modification in the main structure is provided by the fact that a fully buried arch resists blast loading through hoop compression; on the other hand components of the end bulkheads and appurtenant structures resist the blast load wholly or

partially in flexure and strengthening a flexural member is accomplished readily by reducing its span. Many concepts capable of increasing the hardness of the various component structures were considered, but none of them is considered reasonably reliable when one is limited to field modification. Therefore, the various concepts considered will be summarized as well as some of the reasons for not favoring them.

End Bulkhead. By reducing the span of the vertical posts in the end bulkhead to one-half the original value, the moment resistance is increased four-fold provided there is no deformation of the supports. Reducing this span could be accomplished by (a) providing two bulkhead trusses at each end; one at the level of the sill as in the basic design and one at mid-height; (b) providing concrete dead men and cables which support the posts at mid-height; and (c) tying with cables the center of the posts to utility structures placed at right angles to the passageway (cf Fig. H-1). Alternate (a) was discarded because it would also require placing a second set of sills for the full length of the structure at mid-height to carry the reactions of the truss and because the second truss would interfere with any passageway placed at the end of the structure. Alternate (b) was considered undesirable since dead men and cables classically are difficult to arrange to give effective support without attendant deformations and the deformations must be severely limited to insure providing the resistance required; of secondary importance was the relatively complex connections required between the posts and cables. Alternate (c) was considered undesirable also for the same reason but primarily because premature failure of either the end bulkhead or the utility structure doubtless would trigger failure of the other component.

Another alternative, of course, consists of using a greater number of posts and modifying the wood blocks spanning between them by cutting them in half or in thirds in the field and in modifying the bulkhead truss to support the added posts at panel points. This alternative seemed undesirable since (a) to obtain a resistance consistent with a hardness of 300 psi which seems possible in the main structure requires three times the number of vertical posts provided in the main structure; and (b) field modification of the truss would require procuring steel which was not a part of the standard system and a relatively large amount of field welding.

These observations led to the tentative conclusion that it would be more economical and would provide as sound a "structural system" if no bulkhead were used at all and the main structure were extended one additional panel at each end to allow compacted backfill to extend within the structure at a slope of perhaps 1 on 1. This earth would form the end "bulkhead." However, there is no completely reliable method for predicting the behavior of this earth bulkhead under blast loads. Consequently, it is a tentative suggestion which must be investigated in detail before it is adopted in the system. This suggests also an interesting possibility of providing an entranceway. Since the entranceway need provide only a means of ingress (egress may be accomplished by digging away the earth bulkhead) the entranceway perhaps could be designed such that it provides no intentional hardness so long as when it failed it did not allow significant blast pressures to enter the structure nor trigger the failure of nearby components such as ventilation structures. The latter requirements are not too difficult to satisfy, but it is beyond the scope of the current effort.

Passageway and Utility Structures. Field modification of the passageway and utility structures presents a problem similar to the one just discussed relative to the end bulkheads, yet it is not as acute. Although the frames which support the sheeting and covers are subjected to direct stress and flexure in combination, their overall resistance can be increased to be capable of achieving approximately a 300 psi hardness by spacing them at 1'-6" instead of 3'-0". To accomplish this spacing requires drilling holes through the steel sills to accept the added frames, a somewhat difficult task but not an insurmountable one to accomplish in the field. By reducing this spacing the resistance of both the timber wall sheeting and the steel covers is increased by approximately four-fold. However, use of one-half of the original spacing for the frames requires inverting the steel covers so that the added frame does not interfere with the structural tees used in the steel covers. This requirement will degrade the resistance of the covers since the tees will carry the compressive stress induced by flexure, and as a result they will be susceptible to lateral buckling which is not encountered when the covers are placed in the manner originally intended. Nevertheless the resistance of these covers will not be reduced below that required to achieve a 300 psi hardness.

Another problem introduced by reducing the spacing of the passageway (and utility) structure frames results when the connection of the vertical entranceway to the passageway is considered. The added frame in this region can be eliminated if wall sheeting can also be eliminated. Wall sheeting can be removed in the single panel of the passageway where the entranceway connects if an arrangement of the passageway and utility

structures is modified to the configuration shown in Fig. H.1. This arrangement, of course, requires that two utility structures be used with each passageway and two additional bulkheads for the passageway (in addition to those supplied in the standard structure) must be provided. Also each utility structure must be provided with twice the number of frames as provided for the standard (100 psi) structure.

Each end bulkhead used in the passageway and utility structure for the structures with increased hardness must be reinforced to give a resistance compatible with the larger loading. Increasing this resistance is difficult, and it may be desirable (and more economical) to extend the structure and use the earth bulkhead suggested previously for the main structure.

Entranceway and Ventilation Structures. Evaluation of the behavior of the entranceway under blast loading is a highly indeterminate problem: (1) The reaction from the domed closure (hatch cover) would be distributed through the support to both the soil and the pipe columns; (2) after some deformation of the soil and pipe columns occurs, the actual concrete and corrugated steel entrance pipe doubtless would resist part of the reaction; and (3) the support for the domed closure doubtless partially shields the entrance pipe from the direct blast force, but at the same time this support produces forces which act on the entrance pipe. Although it is impossible to make a rigorous analysis of the general behavior, it is apparent that sufficient strength can be obtained by provision of a third standard corrugated pipe with an inside diameter of 5'-0" (This diameter is dictated by the spacing of the pipe columns supporting the hatch and not by strength.) around the entire entranceway and filling the

annular space between the standard entrance pipe and the added corrugated pipe with concrete. The added corrugated pipe would be a special item which is not included in the standard structure. It should be noted that the thickness of the corrugated pipe is not critical since the concrete fill provides the requisite strength.

The domed hatch cover will resist the loading associated with an overpressure of 300 psi because the thickness of it was increased to much more than three times the theoretical thickness in the standard design to allow welding of hinges and sealing angle to it.

Because of the necessity of spacing the frames in the passageway and utility structures at one-half their designed spacing to accomplish the increased hardness, it would be necessary to use only the cover, PC-3, in an inverted position to support the ventilation tube. This also would necessitate field drilling of holes in the channels which support the ventilation tube. Furthermore, a special connection must be provided to attach the intake duct to the ventilation tube which will allow the top member of the added frame to pass through the connection. The concrete encasement provided by Case B (See Design Drawings) would appear to be adequate in reinforcing the ventilation tube to resist an overpressure of 300 psi.

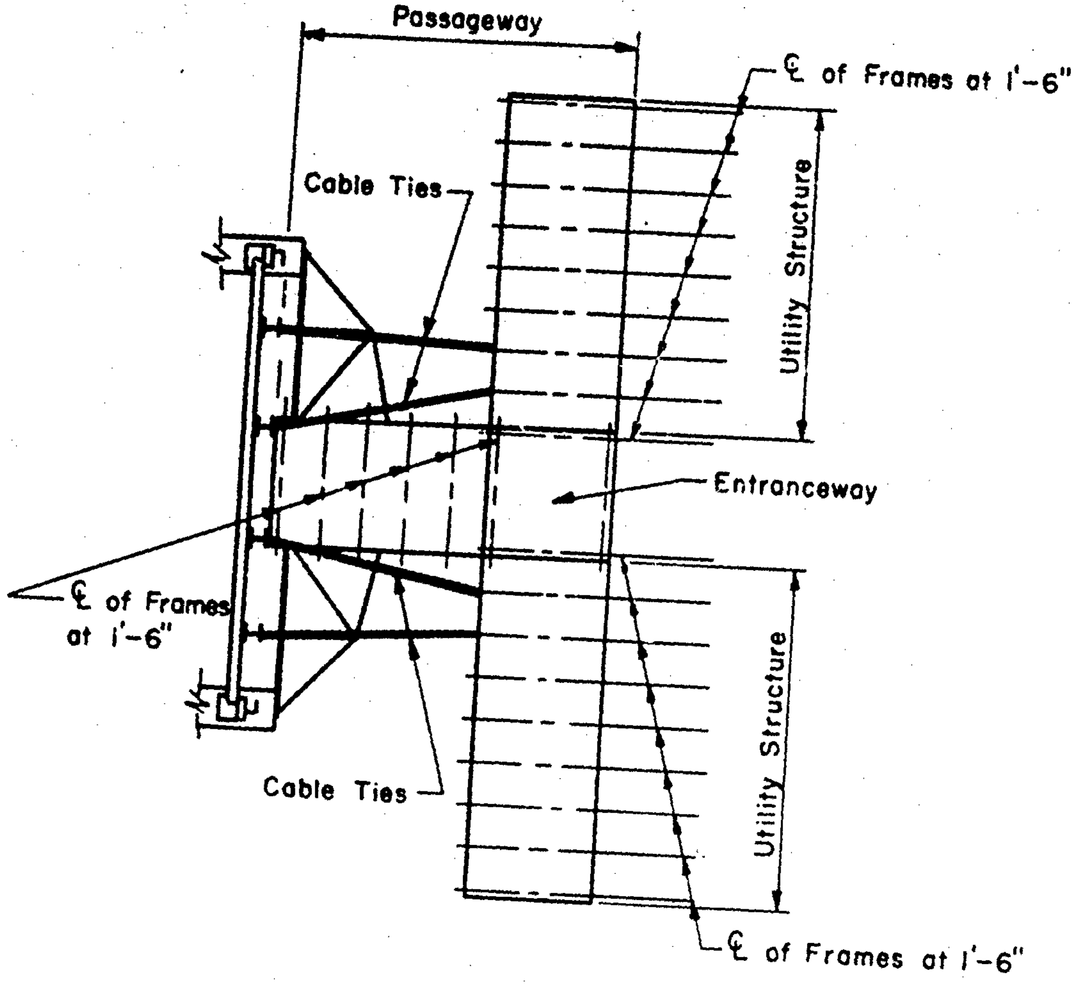
MODIFICATION FOR RADIATION PROTECTION

If the structure is to provide protection at ranges consistent with a side-on overpressure of 300 psi provision of the increased protection factors against radiological hazards becomes a significant problem. However, approximate analysis of the type presented in Appendix D indicate that the radiological dose within the structure can be held to approximately

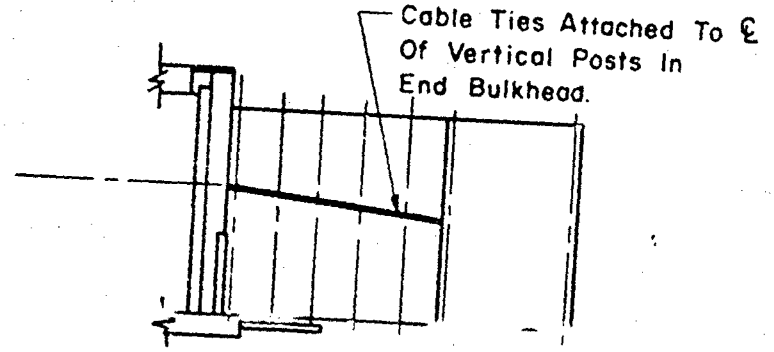
100 REM if an additional one foot of cover (increasing the cover over the crown of the arch to 5 ft.) is provided even for a range consistent with a 300 psi overpressure.

SHOP MODIFICATION

Of course the kits may be readily modified to provide a structure with 300 psi resistance initially, using the methods and principles described herein. This is not provided for in the drawings and instructions, however. Nevertheless no changes in the basic concepts are necessary to provide the stronger structure.



PLAN



ELEVATION

FIG. H.1 SCHEMATIC CONFIGURATION OF PASSAGEWAY AND UTILITY STRUCTURES FOR POSSIBLY INCREASING HARDNESS.

APPENDIX I

MISCELLANEOUS ALTERNATIVE STRUCTURAL CONFIGURATIONS

There are summarized in Chapter 3 the specific alternative means of construction considered during the process of designing the basic shelter described in the main report Hence, this appendix describes briefly those concepts which show promise for the specific type of construction considered and for other associated types of construction. Several obvious alternatives, such as the substitution of a reinforced concrete arch for the structural tees and timber blocks, will not be discussed.

ALTERNATES FOR STANDARD STRUCTURE

Following completion of the design of the passageway and utility structures, it became apparent that a simple addition to the kit containing the utility structure would allow it to be arranged with its axis perpendicular to the axis of the passageway or other utility structures. Fortuitously, the span of the steel frames, which was dictated by the clearance between vertical posts in the end bulkhead of the main structure, corresponds ideally with the spacing between frames. Thus, without seriously restricting the clearances, utility structures may be framed into the sides of other utility structures (or passageway) by merely providing an opening in the timber lagging and by providing an additional steel frame in the utility structure to support the timber lagging and steel covers at its point of intersection with another utility structure. A positive connection between structures arrayed as a "T" can be accomplished by drilling holes in the steel frames at the intersection and bolting them together.

The advantage of the arrangement just discussed is that it provides great latitude in arranging equipment in these appurtenant structures, and possibly of greater importance, the passageway and utility structures can be used as connections between main structure installations which may be arrayed side-by-side, end-to-end, or perpendicular to one another. Being able to connect main structure installations to one another easily may prove advantageous for purposes of direct communication, for using one ventilation system to service several shelters, and possibly for increased morale among shelter occupants. It should be noted in this regard that, although the passageway and utility structures are shown attached to only one end of the shelter in the detailed plans, a passageway can be connected to either end or to both ends of the main structure.

Also it appears quite possible that the utility structure, at least, could be fabricated from an elliptically shaped or oval corrugated steel structure (frequently referred to as a cattlepass). This concept was discarded in the actual design of the shelter because: (1) it is difficult to frame the entrance tube into the elliptical section and (2) the stability of the relatively flat surfaces was questionable. If the rectangular passageway as designed is used to attach the vertical entrance tube while cattlepasses are used for utility structures and connections between shelters, the first objection can be circumvented. The problem of stability of the relatively flat wall surfaces of the oval structure can be solved by bolting a steel beam, as a wale, longitudinally at the center of the flatter wall surface and by placing temporary struts between opposing wales. This is similar to the temporary struts specified for the steel frames used in the basic design.

SANDWICH CONSTRUCTION

Relatively early in the consideration of possible structural configurations appropriate for use as a shelter, a type of construction was conceived which offers great promise for increasing the hardness (or also the static strength) of existing arched structures and potentially a means of providing extreme hardness economically. This type of construction was not adopted for the basic design because it appeared too sophisticated a system to be erected in a forward area in a brief period of time.

Sandwich construction has been adopted as a name for the proposed concept, and this name graphically describes the cross-section through the arch. The construction would begin by forming an appropriate foundation and erecting on it in standard fashion a standard corrugated metal, steel plate, or reinforced concrete arch of the requisite span. A few inches outside of this arch, and concentric to the first erection of a second standard arch is begun. The spacing between the arches is dictated by the clearance required to later fill the space with another material, and partially by the strength required. Spacers are fastened to both the inner and outer arches as the second arch is erected. Alternatively shear lugs can be fastened to each arch separately before erection of the second arch begins. As erection of the second arch proceeds, or after completion of erection of the second arch except for plates at the crown, the space between the two arches is filled with sand, soil, soil cement, concrete, or another appropriate material. This filling material, being confined in the annular space, will develop much larger strengths than would be implied by its unconfined properties. Obviously this procedure can be used to increase the resistance (both static and dynamic) of existing

arched structures of any material. The footings can be extended and an arch erected outside of the existing structure and the annular space filled with an appropriate material.

The process of erecting additional arches and filling the space between arches can be continued to provide as large a resistance as desired. In this fashion extraordinary resistance could be gained very economically. Therefore, it is a concept which deserves strong consideration for future structures.

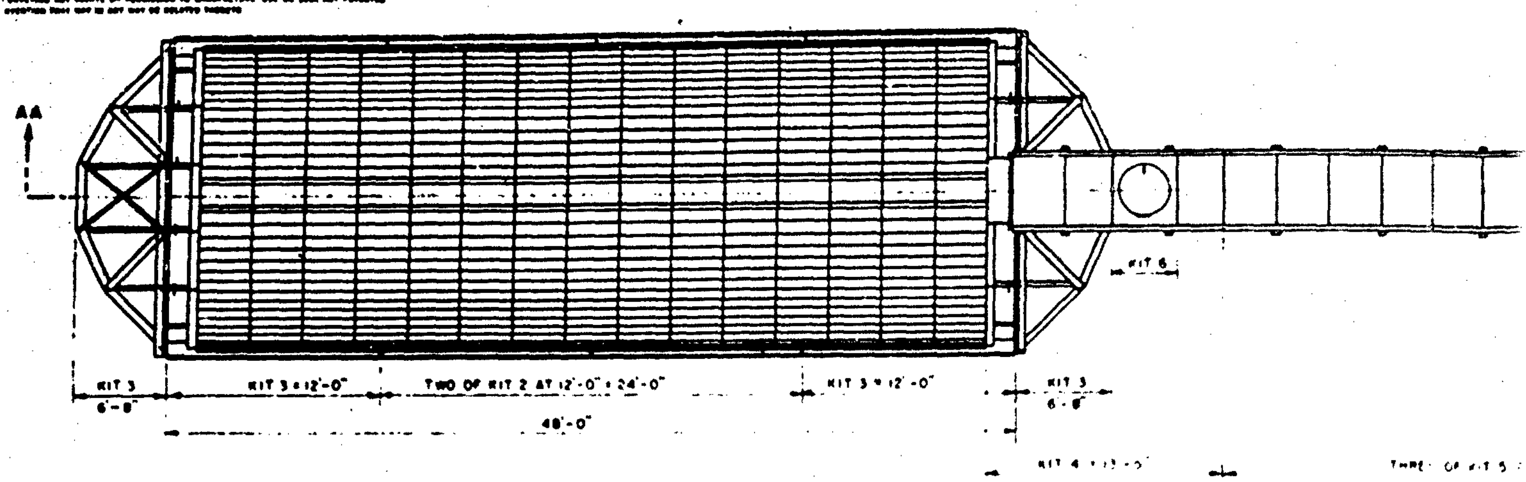
APPENDIX J

DETAILED PLANS

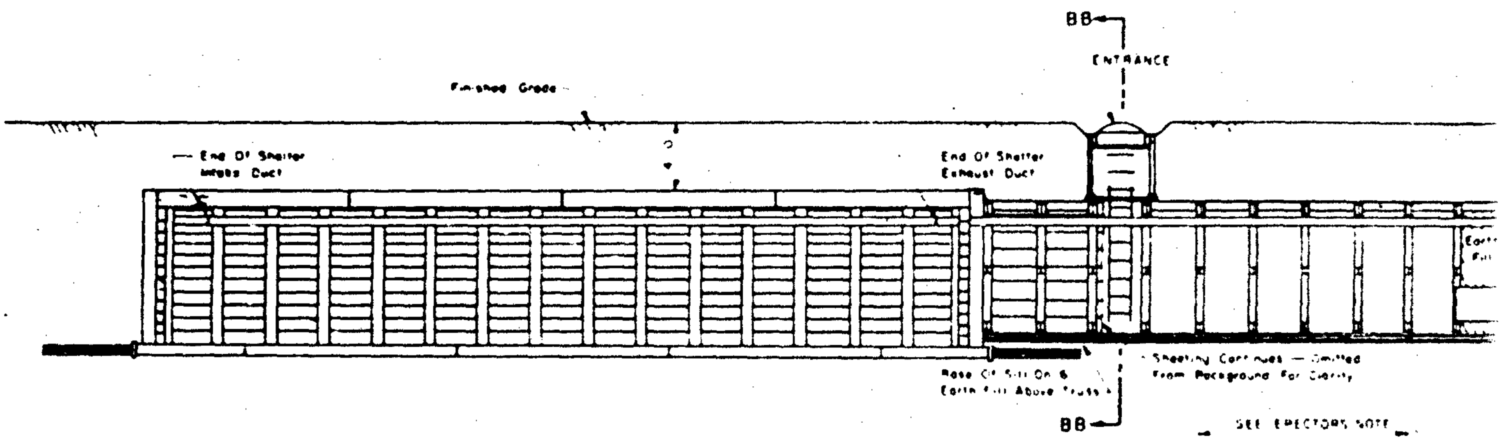
This portion of the appendix contains the detailed plans for the proposed shelter. Included on the plans are detailed fabrication instructions as well as recommended erection procedures.

INDEX OF DRAWINGS

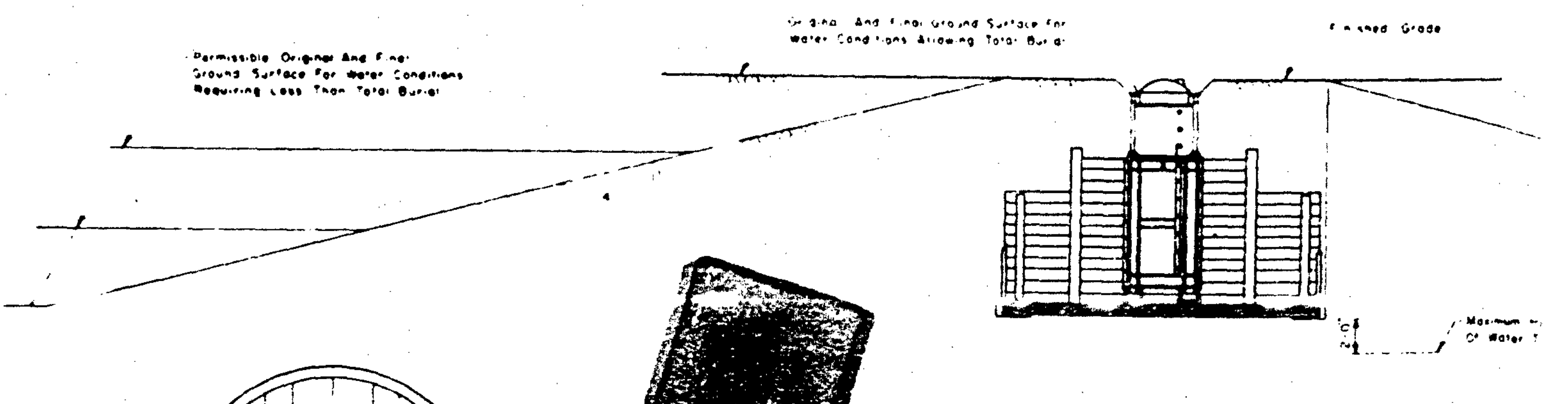
Sheet No.	Title
1	General Layout
2	Pictorial View
3	Main Structure
4	End of Main Structure and Bulkhead
5	Bulkhead Truss
6	Passageway Structure
7	Utility Structure
8	Hatch Assembly
9	Entrance Structure
10	Ventilation Structure



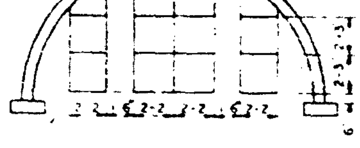
PLAN VIEW — EARTH COVER OMITTED



SECTION AA-AA



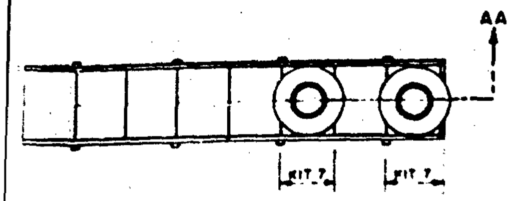
SECTION BB-BB

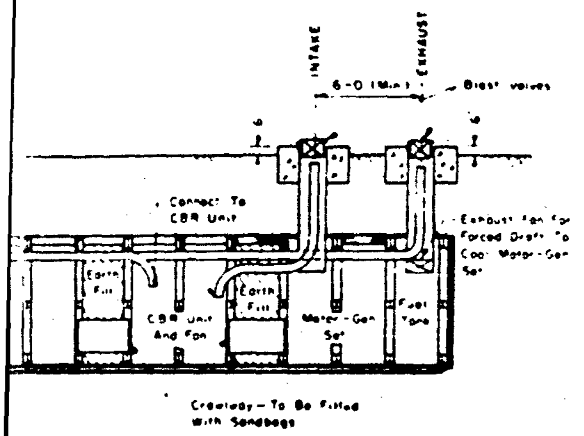


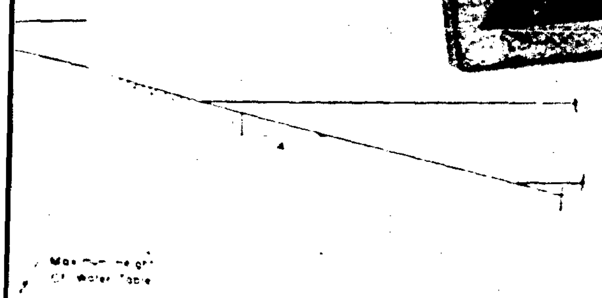
PROPOSED BUNK ARRANGEMENT

ERECTORS NOTE

REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL







GENERAL NOTES

1. Design Loading: 100 psi side-on overpressure.

2. Radiation Protection Factors:

   a. Fallout (gamma): in excess of 20,000.
   b. Prompt or initial gamma: in excess of 5000 for a mean spectral energy of 4 Mev. (Each additional foot of cover over the recommended 4 ft increases protection factor by approximately a factor of 4.)
   c. Neutron: in excess of 20,000.
   d. Thermal: recommended depth of burial adequate for thermal pulse.

3. 100 psi Ranges and Incident Prompt Gamma Doses from Surface or Low-Air Burst:

Yield MT	Range for 100 psi ft	Associated Prompt Gamma R
0.001	350	73,000
0.010	750	651,000
0.100	1630	310,000
1.0	3500	205,000
10	7540	22,800
50	12800	< 20,000

4. Fire Storm Protection: Dependent upon the equipment installed.

5. Occupancy: Basic shelter designed for 51 men for two week's period, assuming one bunk per man. Not suitable for long-time usage unless timber treated with non-toxic preservative.

6. Equipment Provided: Basic shelter provides only structural components and assemblies. All equipment must be provided separately. See Table of Additional Requirements.

7. Equipment and Supplies Installation: All equipment and supplies must be placed in shelter (and in utility structure, if included) before roof is completed.

8. Emergency Exits: Use axe to remove ceiling blocks.

9. Water Table: Structure must be located above water table. Use alternate method of burial with high water table.

10. Ground Water Seepage: All exterior surfaces of shelter and passageway (including utility structure) must be covered with polyethelene sheeting, properly lapped, to prevent ground water seepage.

11. Bunk Installation: To be suspended from arch blocks by screw hooks.

12. Floor: None provided in basic kits, may be plywood or as desired.

MATERIAL SPECIFICATIONS

Structural Steel	ASTM A-7
Standard Nuts and Bolts	ASTM A-307 Grade A. All bolts and nuts, unless otherwise indicated are to be American Standard, regular, hexagon head.
Light Gage Steel	ASTM A-245
High Strength Nuts and Bolts	ASTM A-325, American Standard, regular semi-finished hexagon head bolts with heavy hexagon head nuts.
Dry Pre-Mix Concrete	Portland Cement (TYPE III) ASTM C-150, sand and coarse aggregate (1.4" max) to be premixed and placed in 1 cu. ft bags. Mix to be proportioned for f'_c = 3000 psi at 28 days. Amount of water per bag to be specified on bag.
Polyethelene Sheeting	10 mil; Commercial Standard CS 238-61
Timber	Stress Graded, s4s, Douglas Fir (Coast Region) or Southern Pine or equal with following minimum modulii of rupture.

Moisture	Modulus of Rupture
green	7000 psi
12%	9000 psi

TIGHTENING OF HIGH STRENGTH BOLTS

All high strength bolts to be tightened with the following procedure: (1) First tighten to snug fit to insure that all parts being bolted are in full contact with each other. Snug tight is the tightness attained by full effort of one man using an ordinary spud wrench (without extension). (2) Then tighten all bolts additionally by 1/2 turn of the nut relative to the bolt. Use heavy duty spud wrench and about 4' lever arm.

SEE SHEET 2 FOR SCHEDULE OF KITS

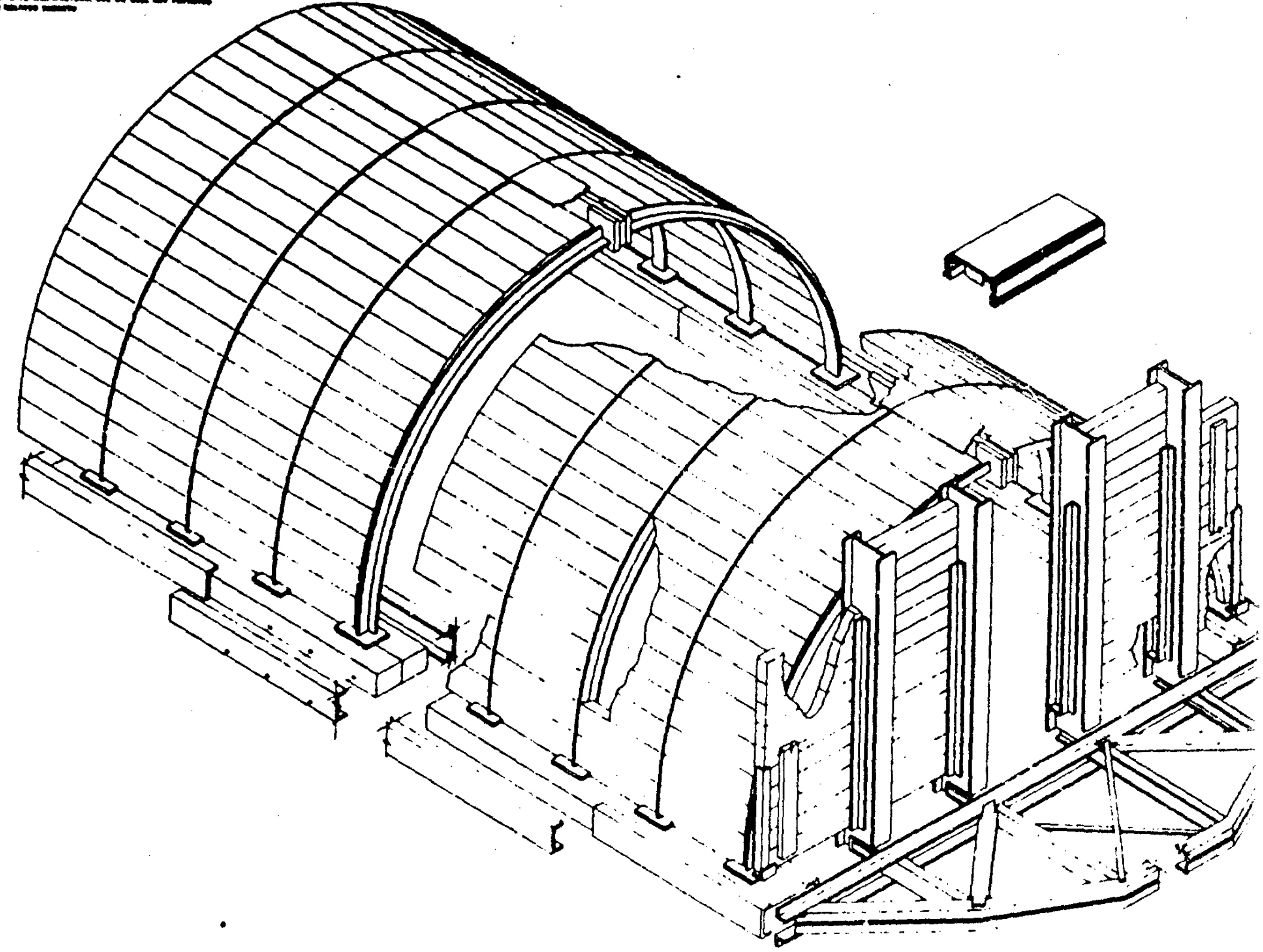
ERECTORS KIT

One or more Utility Structures (Kit 5) may be added in the area shown. These additional structures may be used to house such items as:

1. Decontamination showers and associated equipment
2. Additional vertical entrance (Kit 6)
3. Additional CBR Units
4. (a) Oxygen, Chlorate Candles or similar items and metering equipment
   (b) CO_2 scrubbers
5. Air conditioning equipment and associated cooling reservoir
6. Other desired items

APPROVED DIRECTOR RECOMMENDED DIV. CHIEF SUBMITTED	GENERAL LAYOUT	U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS.
DESIGNER DRAWN / TRACED / CHECKED	PROTECTIVE SHELTER	N. M. NEWMARK Urbana, Illinois ; Contract DA-22-079-eng-225
	SCALE / DATE 1 Sept. 1962	SHEET 1 OF 10

NOTICE - [illegible]



ADDITIONAL SHELTER REQUIREMENTS*

Not provided with Basic Shelter Kit.

- Food and Medical Supplies
- Water (Potable and nonpotable)
- Communication equipment
- Toilet and Waste Disposal
- Bunks, hangers and screw hooks
- Air Conditioning equipment and cooling reservoir
- Flooring materials
- Lighting
- Decontamination showers, tanks and equipment
- Radiation monitoring equipment, exterior and interior
- Oxygen, CO_2 scrubbers
- Periscope and mounting equipment
- Blast valves
- CBR Unit and Fan
- Ventilation, power and associated equipment

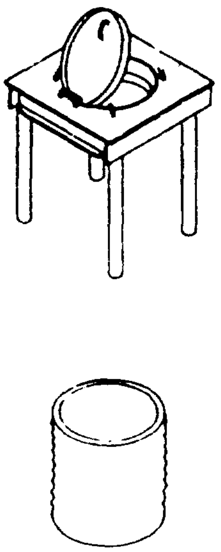
*Some of these items may be omitted for the most austere type shelter

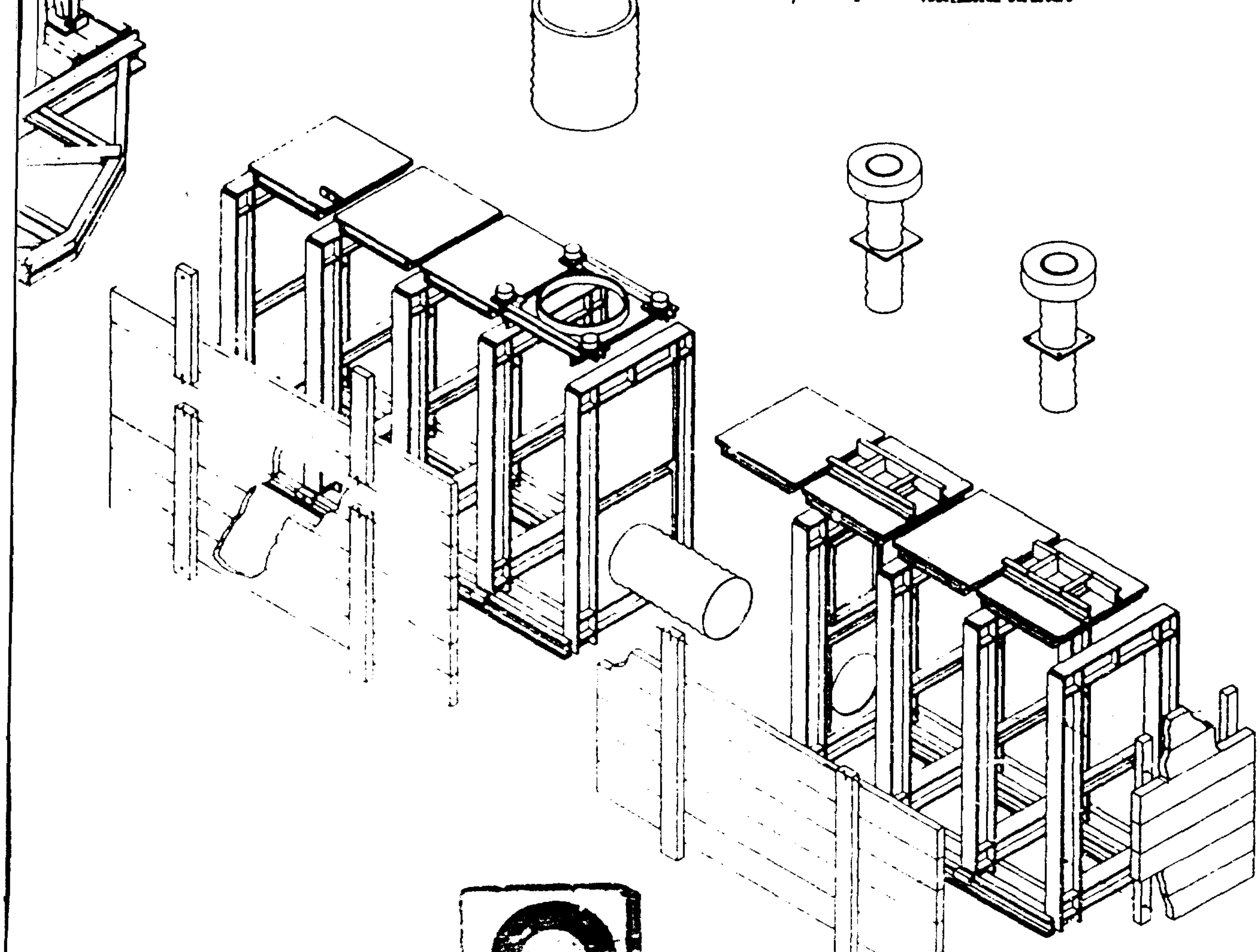
REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL

SCHEDULE OF KITS FOR ONE 31 MAN SHELTER

Basic Shelter

Kit No.	No. of Kits Req'd.	Contents		Sheet No. for Kit
1	1	Erection Kit A-Frame and hoist, mountable on truck or tractor; 1 ton min. capacity; 12 ft. min. reach Assorted tools:		
		Wrecking Bars	Picks	
		Brace and Wood Bits	Shovels	
		Portable Circular Wood Saw	Hammers	
		Flame Cutting Equipment	Sledges	
		Portable Steel Saw (Optional)	Axes	
		Heavy Duty Wrenches and Sockets	Rammers	
			Drift Pins	
			Soil Tampers	
2	2	Main Structure		3
3	2	End of Main Structure and Bulkhead		4 and 5
4	1	Passageway Structure		6
5	2	Utility Structure		7
6	1	Entrance Structure		8 and 9
7	2	Ventilation Structure		10





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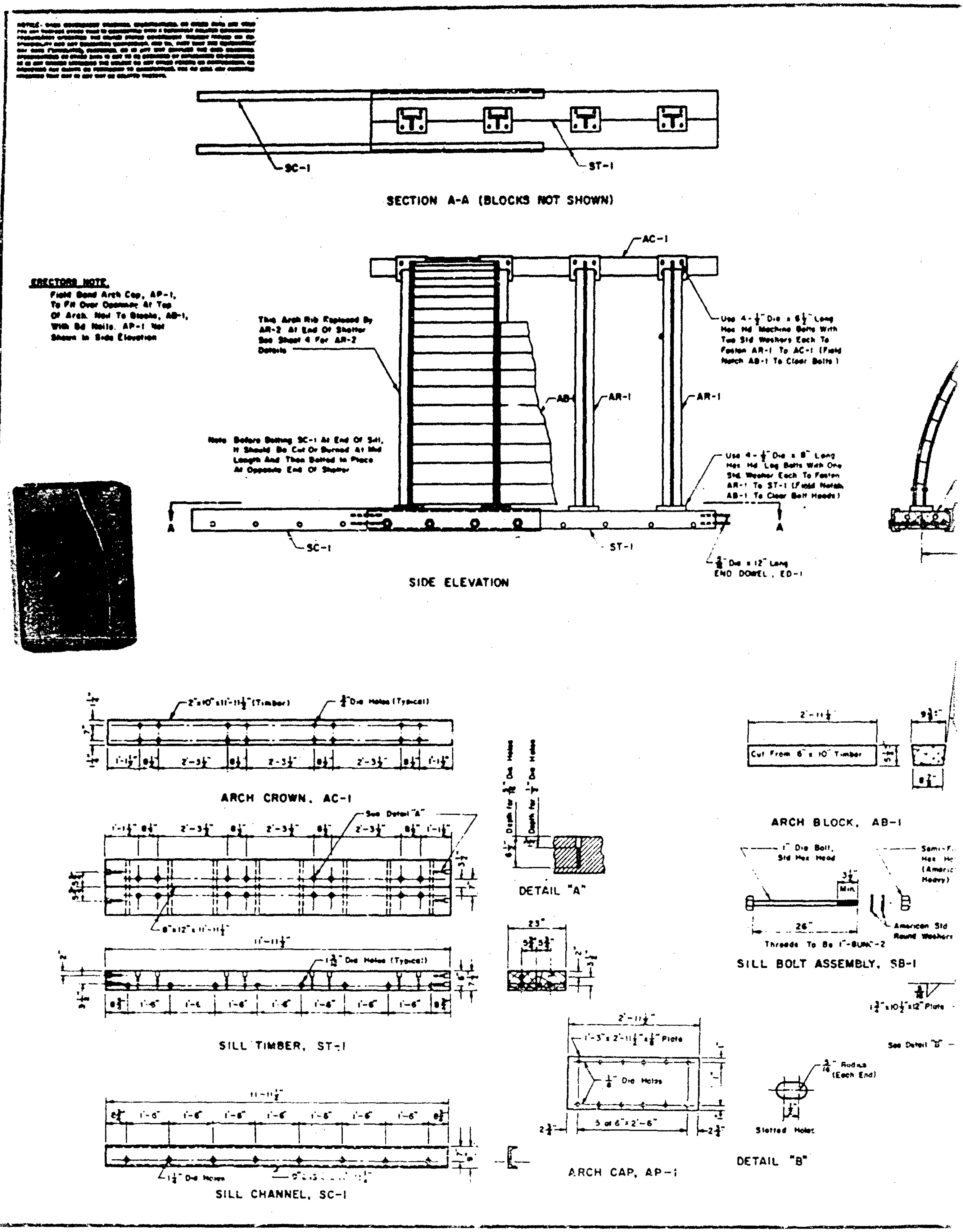
APPROVED DIRECTOR RECOMMENDED DIV. CHIEF SUBMITTED DESIGNER DRAWN / TRACED / CHECKED	PICTORIAL VIEW PROTECTIVE SHELTER	U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-225
	SCALE. / DATE 1 Sept. 1962	SHEET 2 OF 10


SC-1
ST-1
SECTION A-A (BLOCKS NOT SHOWN)
AC-1
ERECTORS NOTE.
Field Bend Arch Cap, AP-1, To Fit Over Opening At Top Of Arch. Nail To Blocks, AB-1, With 8d Nails. AP-1 Not Shown In Side Elevation
This Arch Rib Replaced By AR-2 At End Of Shelter See Sheet 4 For AR-2 Details
Use 4 - 1/2" Dia x 6 1/2" Long Hex Hd Machine Bolts With Two Std Washers Each To Fasten AR-1 To AC-1 (Field Notch AB-1 To Clear Bolts)
AB-1
AR-1
AR-1
Note: Before Bolting SC-1 At End Of Sill, It Should Be Cut Or Burned At Mid Length And Then Bolted In Place At Opposite End Of Shelter
Use 4 - 1/2" Dia x 8" Long Hex Hd Lag Bolts With One Std Washer Each To Fasten AR-1 To ST-1 (Field Notch AB-1 To Clear Bolt Heads)
A
A
SC-1
ST-1
5/8" Dia x 12" Long END DOWEL, ED-1
SIDE ELEVATION
2"x10"x11'-11 1/2" (Timber)
5/8" Dia Holes (Typical)
1'-1 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
1'-1 1/2"
ARCH CROWN, AC-1
See Detail "A"
1'-1 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
2'-3 1/2"
8 1/2"
1'-1 1/2"
8"x12"x11'-11 1/2"
11'-11 1/2"
1 3/16" Dia Holes (Typical)
8 3/4"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
8 3/4"
SILL TIMBER, ST-1
Depth for 1 3/16" Dia Holes
Depth for 1" Dia Holes
DETAIL "A"
23"
5 3/4"
5 3/4"
2'-11 1/2"
Cut From 6" x 10" Timber
ARCH BLOCK, AB-1
1" Dia Bolt, Std Hex Head
3 1/2" Min
26"
Threads To Be 1"-8UNC-2
American Std Round Washers
SILL BOLT ASSEMBLY, SB-1
1/2"x10 1/2"x12" Plate
See Detail "B"
2'-11 1/2"
1'-3" x 2'-11 1/2" x 1/8" Plate
1/4" Dia Holes
2 3/4"
3 at 6" = 2'-6"
2 3/4"
ARCH CAP, AP-1
Radius (Each End)
Slotted Holes
DETAIL "B"
11'-11 1/2"
8 3/4"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
1'-6"
8 3/4"
1 1/4" Dia Holes
SILL CHANNEL, SC-1

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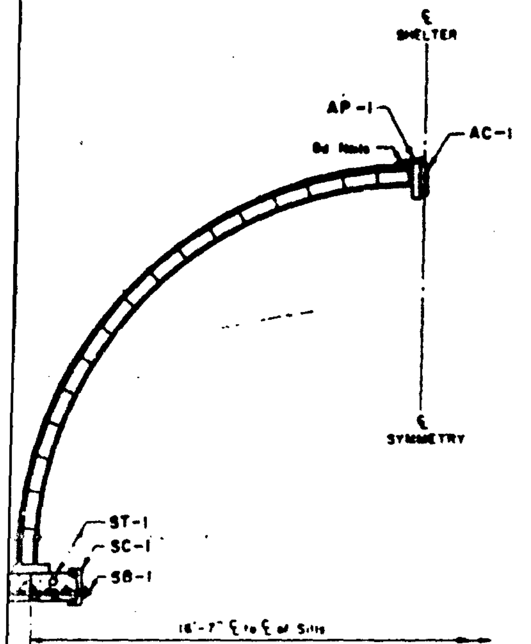
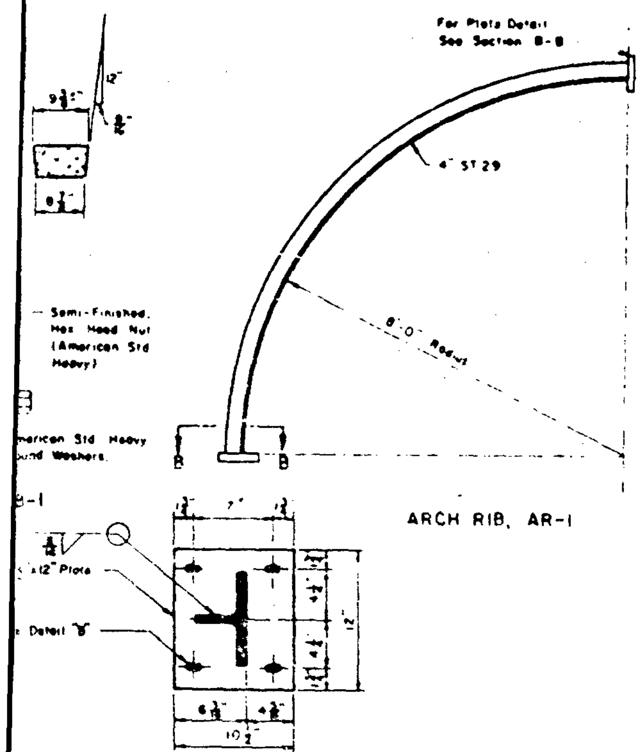


END ELEVATION

BILL OF MATERIALS KIT NO. 2 (MAIN STRUCTURE KIT)

Materials for ONE Kit No. 2 (2 req'd for 51 man shelter)

Piece Mark	Description	No. of Pieces	Unit of Measure	Total	Specifications
STRUCTURAL STEEL AND STEEL FASTENERS			lb.	lb.	
AR-1	Arch Rib	8	490	3920	ASTM A-7 (Welded)
SC-1	Sill Channel	4	160	640	ASTM A-7
AP-1	Arch Cap	4	18.9	76	ASTM A-7 or ASTM A-245
SB-1	Sill Bolts, Nuts and Washers	16	6.6	106	ASTM A-325
ED-1	End Dowels, 5/16" x 12"	8	0.25	2	ASTM A-7
	Hex Hd. Lag Bolts, 1/2" dia x 8"	32	0.44	14	ASTM A-307 Grade A
	Hex Hd. Mach. Bolts and Nuts, 1/2" dia x 6 1/2"	16	0.44	7	ASTM A-307 Grade A
	Washers for 1/2" dia bolts	64	0.04	}	
	8d Nails			}	
	Subtotal, Steel and Fasteners			4768 lbs.	
TIMBER			bm	bm	
ST-1	Sill Timbers	4	192	770	See Sheet 1
AC-1	Arch Crown	1	20	20	
AB-1	Arch Blocks	136	15	2040	
	Subtotal, Timber		mbm	2.8	
			lb.	6400	
	Polyethelene Sheeting, 15' x 50'	1	lb.	56	See Sheet 1
			Weight/Kit	5.6 TONS	



ARCH RIB, AR-1

SECTION B-B

ERECTION PROCEDURE

1. Assemble both sills:

   a. Place polyethelene sheeting on subgrade such that longitudinal centerline of shelter bisects the 50' sheet length. Unroll sheets to permit sill to rest on sheet in trench.
   b. For each sill, place sill timbers (ST-1) side-by-side in approximate location.
   c. Insert end dowels (ED-1) in holes in end of each sill timber.
   d. Place two more sill timbers (ST-1) at each end of the members placed in Step 1a using end dowels (ED-1) to align timbers.
   e. Splice sill timbers (ST-1) with sill channels (SC-1) using sill bolts (SB-1).
   f. Align each sill so that the center to center distance is 16'-7". Both sills to be level and at same elevation. Sills must be positioned such that the arch ribs (AR-1 and AR-2), when installed, will be perpendicular to centerline of sill.
   g. Continue Steps 1c through 1f until both sills are completely assembled. Final assembly will require cutting 2 sill channels (SC-1) at the midpoint. For each complete sill, the half lengths of SC-1 are bolted in place at the ends of the shelter.

2. Position end trusses (ET-1), at ends of sills. Side marked "top" must be up. Trusses should be leveled by timber blocks as required. Use 1/2" dia lag bolts to attach trusses to sill.

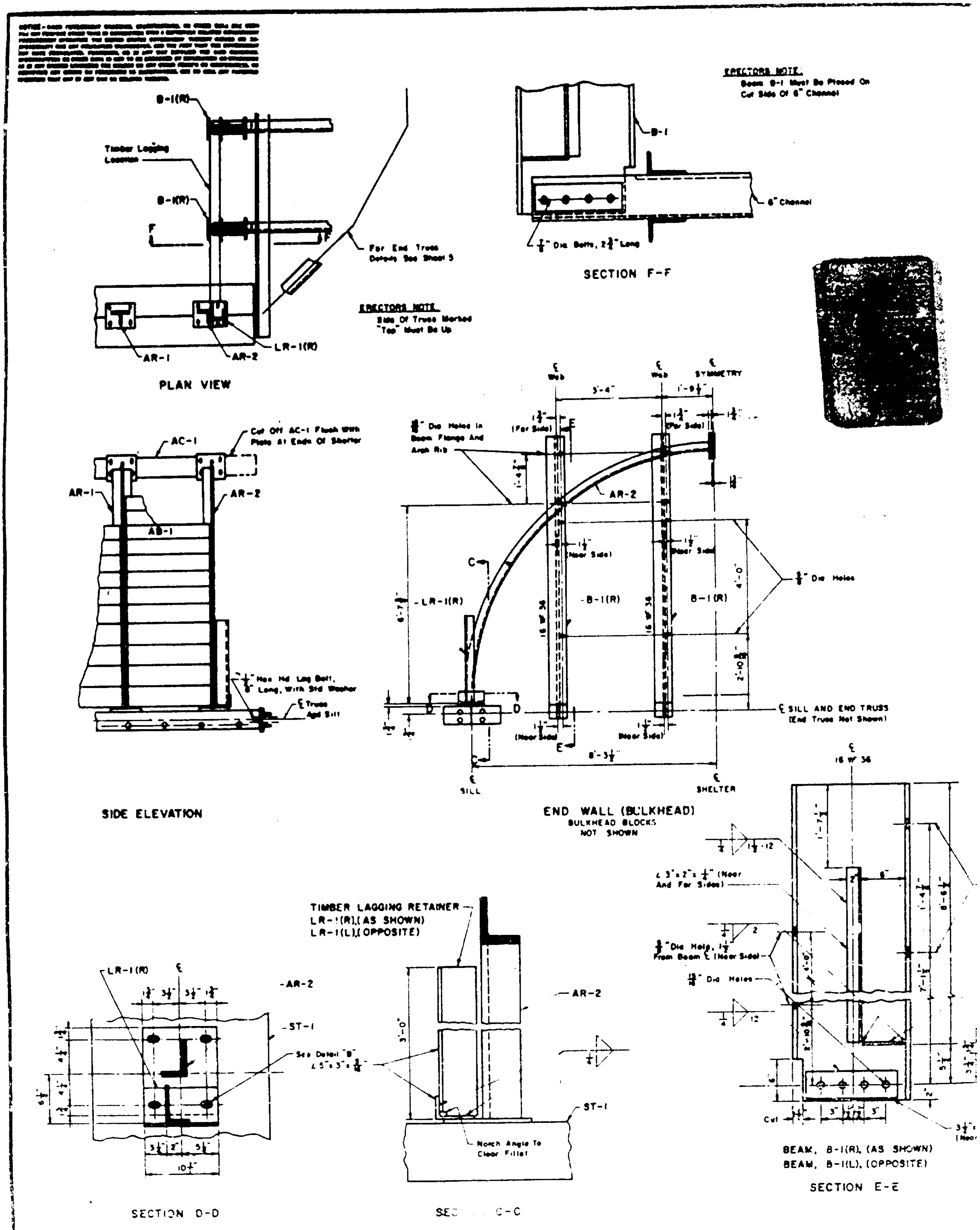
3. Place arch ribs and arch blocks:

   a. At the passageway end of the shelter, place two opposing arch ribs, (AR-2), in place and bolt loosely with two 1/2" dia lag bolts each. Make sure that upright leg of angle is at shelter end. Insert arch crown, (AC-1) between the ribs at top of arch and bolt loosely with one 1/2" dia machine bolt. Before tightening the lag screws that fasten AR-2 to sill, lagging retainer (LR-1) must be positioned. Note right and left for LR-1.
   b. Erect two opposing arch ribs (AR-1) at free end of arch crown (AC-1). Bolt AR-1 to sill with two 1/2" dia lag bolts and join the tops of AR-1 to AC-1 with 1/2" dia machine bolts. Erect and bolt intervening AR-1s. Complete bolting of Step 3a above.
   c. Continue erecting arch ribs (AR-1 and AR-2) and simultaneously place arch blocks (AB-1) in position between arch ribs. First and last arch block between each pair of ribs must be field notched to clear bolts.
   d. Place arch cap (AP-1). Field bend AP-1 to fit crown of arch. Nail in place with 8d nails.

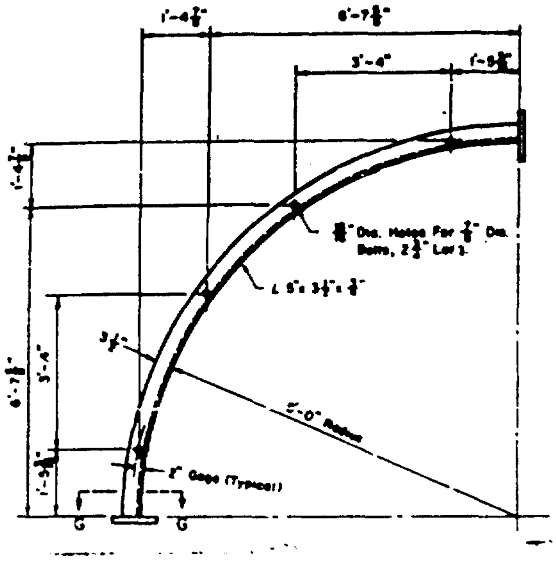
   WARNING: All equipment and materials (including flooring, if used) must be placed within main structure before all of the arch ribs are placed or before the end bulkheads are erected.

(Continued on Sheet 4)

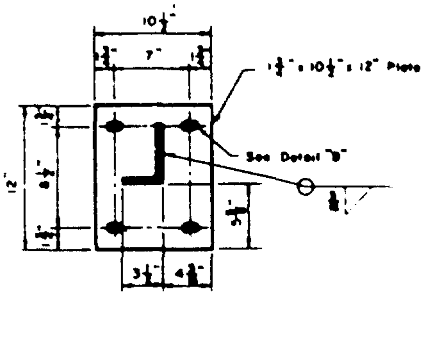
APPROVED ______ DIRECTOR RECOMMENDED ______ DIV CHIEF SUBMITTED [signature] [signature] [signature] DESIGNER [signature]	MAIN STRUCTURE KIT 2	U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS
	PROTECTIVE SHELTER	N M NEWMARK Urbana, Illinois Contract DA-22-079-eng-225
DRAWN [signature] TRACED T.C. CHECKED [signature]	SCALE ______ DATE: 1 Sept 1962	SHEET 3 OF 10


ERECTORS NOTE.
Beam B-1 Must Be Placed On Cut Side Of 6" Channel
B-1(R)
Timber Lagging Location
For End Truss Details See Sheet 5
ERECTORS NOTE.
Side Of Truss Marked "Top" Must Be Up
AR-1
AR-2
LR-1(R)
PLAN VIEW
B-1
6" Channel
SECTION F-F
AC-1
Cut Off AC-1 Flush With Plate At Ends Of Shelter
AB-1
SIDE ELEVATION
SYMMETRY
3'-4"
1'-9 1/2"
(Far Side)
(Near Side)
16 W 36
B-1(R)
LR-1(R)
6'-7 1/2"
4'-0"
8'-3 1/2"
SILL
SHELTER
SILL AND END TRUSS
(End Truss Not Shown)
END WALL (BULKHEAD)
BULKHEAD BLOCKS NOT SHOWN
TIMBER LAGGING RETAINER
LR-1(R),(AS SHOWN)
LR-1(L),(OPPOSITE)
ST-1
Notch Angle To Clear Fillet
SECTION D-D
SECTION C-C
BEAM, B-1(R), (AS SHOWN)
BEAM, B-1(L), (OPPOSITE)
SECTION E-E

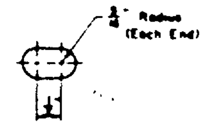
REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL



ARCH RIB, AR-2



SECTION G-G



DETAIL "B"

BILL OF MATERIALS KIT NO. 3 (END BULKHEAD)

Materials for ONE Kit No. 3 (2 req'd for 51 man shelter)

Piece Mark	Description	No. of Pieces	Unit of Measure	Total	Specifications
STRUCTURAL STEEL AND MISC. FASTENERS			lb.	lb.	
AR-1	Arch Rib	6	490	2940	ASTM A-7 (Welded)
AR-2	End Arch Rib	2	375	750	ASTM A-7 (Welded)
SC-1	Sill Channel	4	160	640	ASTM A-7
AP-1	Arch Cap	4	18.9	76	ASTM A-7 or ASTM A-245
SB-1	Sill Bolts, Nuts and Washers	16	6.6	106	ASTM A-325
ED-1	End Dowels, 5/16" x 12"	8	0.25	2	ASTM A-7
BT-1	Bulkhead Truss	1	1090	1090	ASTM A-7 (Welded)
LR-1(R)	Timber Lagging Retainer	1	32	32	ASTM A-7 (Welded)
LR-1(L)	Lagging Retainer	1	32	32	
B-1(R)	Vertical Posts	2	407	814	ASTM A-7 (Welded)
B-1(L)	Vertical Posts	2	407	814	
	Hex Hd Lag Bolts, 1/2" dia x 8"	36	0.44	16	ASTM A-307 Grade A
	Hex Hd Mach Bolts and Nuts, 1/2" dia x 6 1/2"	16	0.44	7	ASTM A-307 Grade A
	Washers for 1/2" dia bolts	52	0.06	3	
	High Strength Bolts and Nuts 7/8" dia x 2 3/4"	20	0.95	19	ASTM A-325
	Washers for 7/8" dia bolts	40	.15	6	ASTM A-325
	Nails for Scab Plate, 16d			1	
	Nails for Arch Cap, 8d			1	
	Subtotal, Steel and Fasteners			6508	
TIMBER			bm	bm	
ST-1	Sill Timber	4	192	770	See Sheet 1
AC-1	Arch Crown	1	20	20	
	Scab Plate	2	6	12	
AB-1	Arch Blocks	136	15	2040	
BB-2	End Bulkhead Blocks	46	13	598	
BB-1	Center Bulkhead Blocks	12	14	168	
BB-3	Top Bulkhead Blocks	2	9	18	
	Subtotal, Timber		mbm	3.6	
			lb.	8800	
POLYETHYLENE SHEETING					
	One - 15' x 50' (floor and outer surface of arch)			36 lb.	See Sheet 1
	One - 12' x 20' (end bulkhead)			12 lb.	
	Subtotal, Polyethylene			48 lb.	
			Weight/Kit	7.4 TONS	

ERECTION PROCEDURE (cont)

4. Erection of end bulkheads and/or passageway may begin as soon as the end arch ribs (AR-2) have been erected per Step 3. See Steps 2, 7, 8 and 9 for details of end bulkhead erection and Sheet No. 7 for passageway erection. Vertical posts (B-1) for end bulkhead must be attached to AR-2 before arch blocks (AB-1) are placed on AR-2.

5. Wrap ends of polyethylene sheeting over the top of the arch.

6. Backfill of main structure must be hand tamped in approximate 6" lifts. Keep backfill at same level on both sides of structure. Avoid driving motorized equipment over the partially backfilled arch.

7. As soon as arch ribs (AR-2) are securely bolted, per Step 3, the vertical beams (B-1) are positioned and fastened to arch ribs (AR-2) and to the protruding truss members with 7/8" dia x 2 3/4" bolts. Note: These bolts are high strength steel bolts.

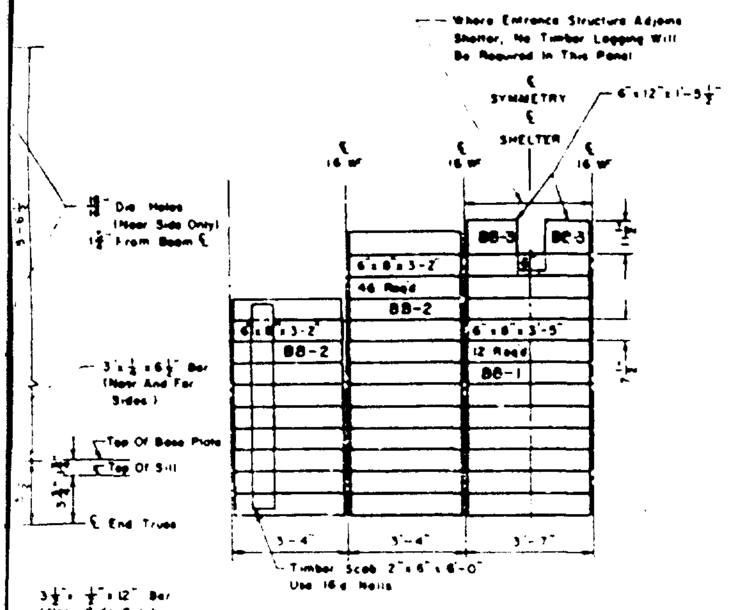
8. Slide bulkhead blocks (BB-1 and BB-2) into position between the vertical beams (B-1) and between B-1 and the lagging retainer (LR-1). Bulkhead blocks to be omitted from center panel at passageway end of shelter.

9. Nail timber scab adjacent to lagging retainer to hold upper blocks (BB-2) in position. Place block BB-3 in position and toe-nail to lower blocks.

10. Place polyethylene sheeting over the end bulkhead without passageway. Care must be taken to see that sheeting is pressed in around vertical beams (B-1) so that it does not bridge between flanges of the beams. Place top sheeting over the arch ribs.

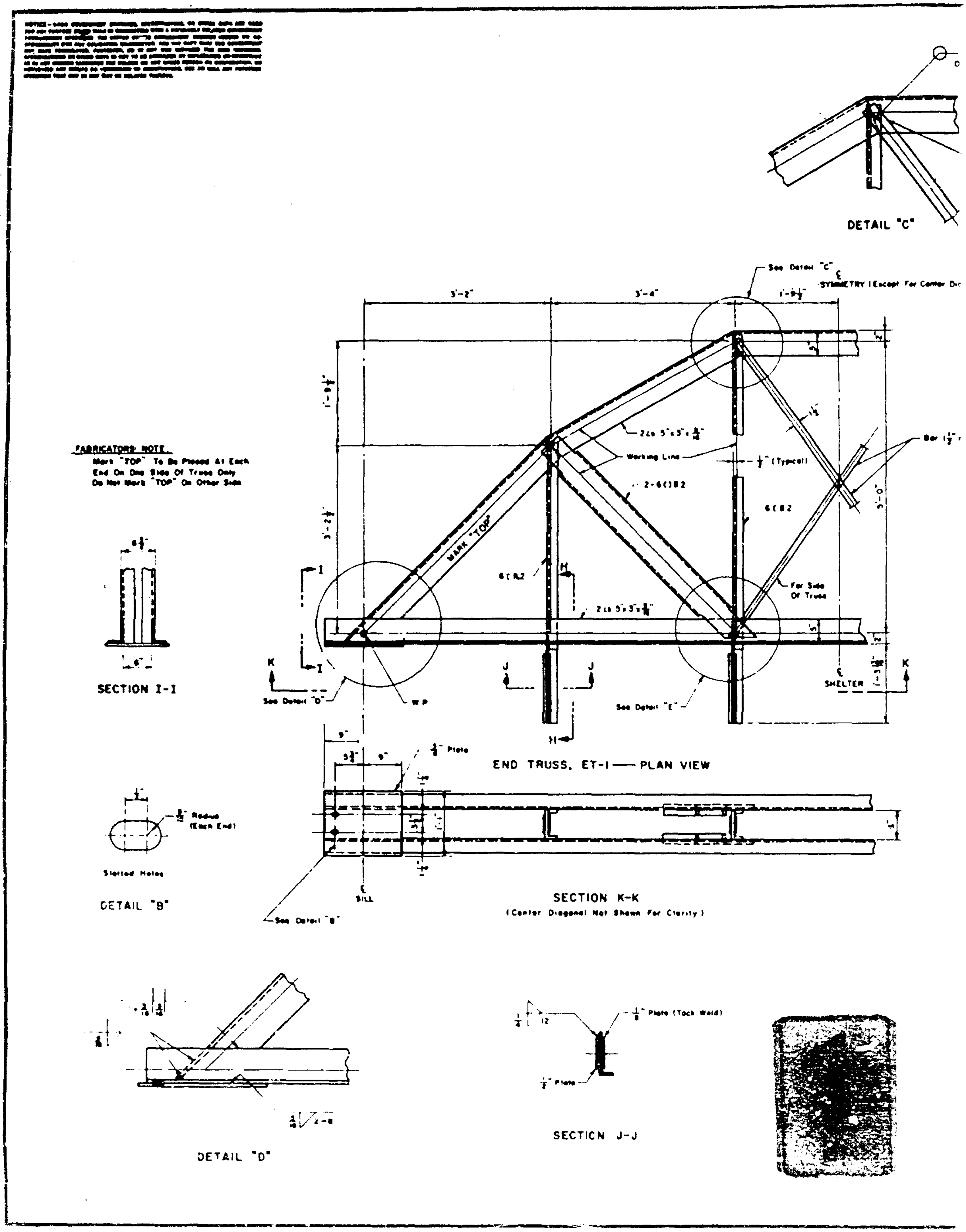
11. Backfill bulkhead end without passageway by hand tamping in 6" lifts.

(Continued on Sheet 7)



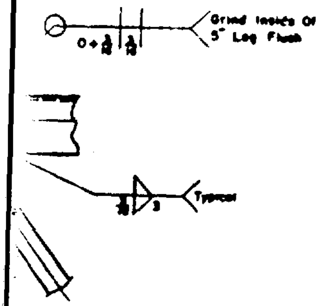
TIMBER LOCATIONS, END BULKHEAD

APPROVED DIRECTOR	END OF MAIN STRUCTURE AND BULKHEAD KIT 3	U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
RECOMMENDED DIV CHIEF		CORPS OF ENGINEERS
SUBMITTED	PROTECTIVE SHELTER	VICKSBURG, MISS.
DESIGNED DRAWN TRACED CHECKED		N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-225
	SCALE DATE 1 Sept 1962	SHEET 4 OF 10



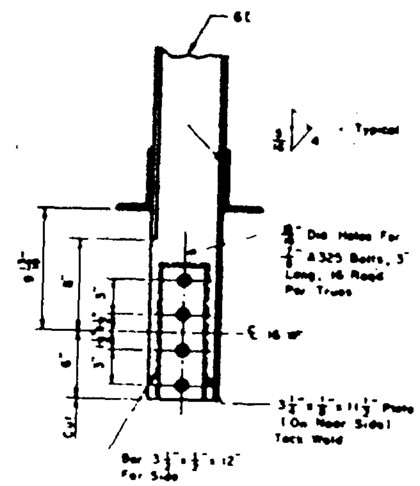
DETAIL "C"
See Detail "C"
SYMMETRY (Except For Center Di
3'-2"
3'-4"
1'-9½"
1'-9½"
3'-2½"
2Ls 5"x3"x 5/16"
Working Line
½" (Typical)
2-6[8.2
6[8.2
6[8.2
Bar 1½"
5'-0"
MARK "TOP"
Far Side Of Truss
2Ls 5"x3"x 5/16"
FABRICATORS NOTE
Mark "TOP" To Be Placed At Each End On One Side Of Truss Only
Do Not Mark "TOP" On Other Side
SECTION I-I
See Detail "D"
W.P.
See Detail "E"
SHELTER
END TRUSS, ET-1 —— PLAN VIEW
⅜" Plate
9"
9"
Radius (Each End)
Slotted Holes
DETAIL "B"
SILL
See Detail "B"
SECTION K-K
(Center Diagonal Not Shown For Clarity)
⅛" Plate (Tack Weld)
½" Plate
SECTION J-J
DETAIL "D"

REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL

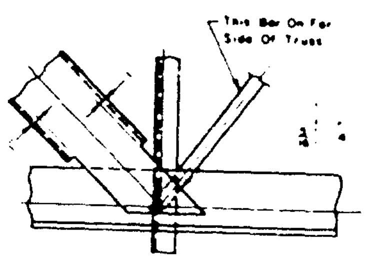


enter Diagonals)

or $1\frac{1}{2}" \times \frac{1}{2}"$



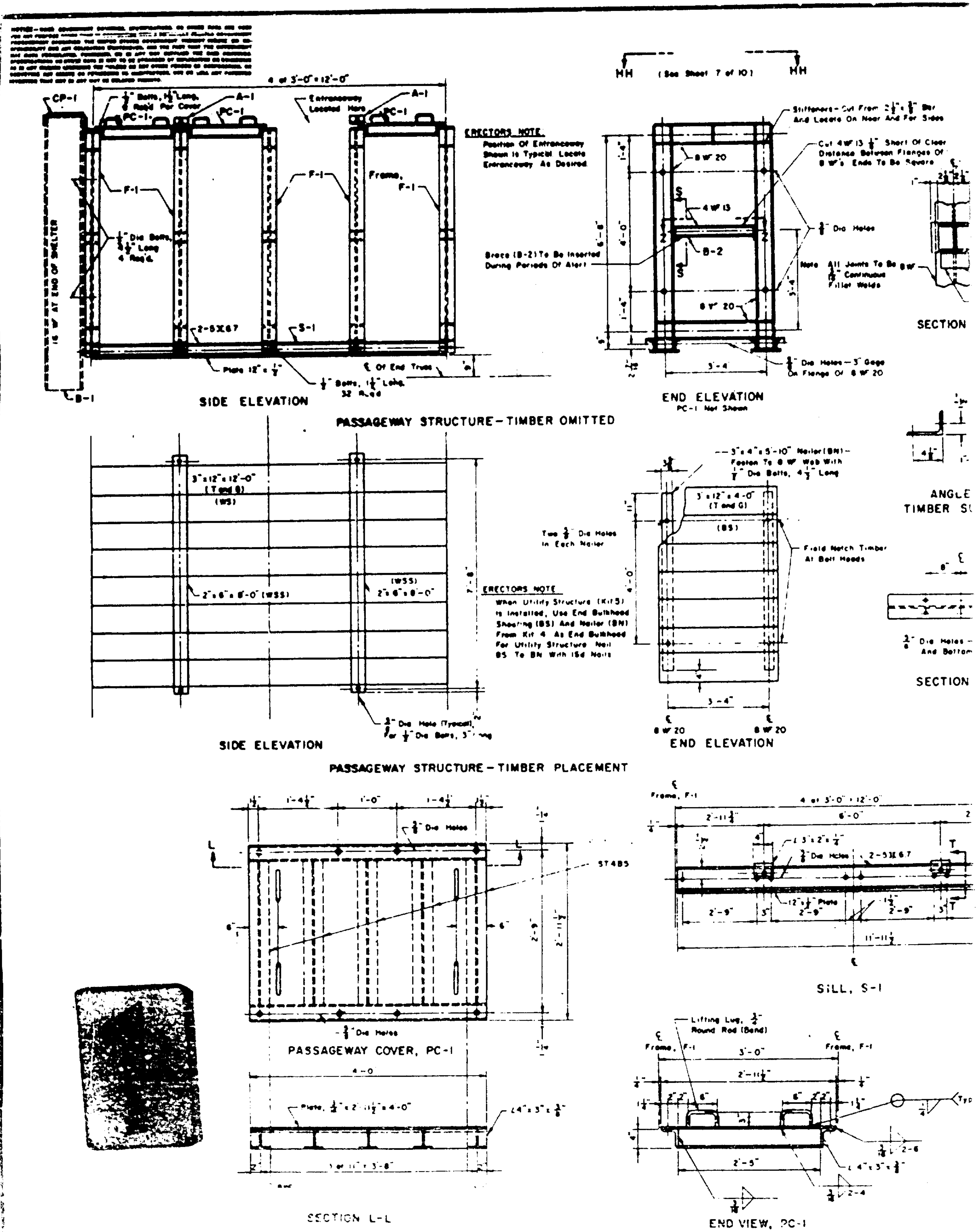
SECTION H-H

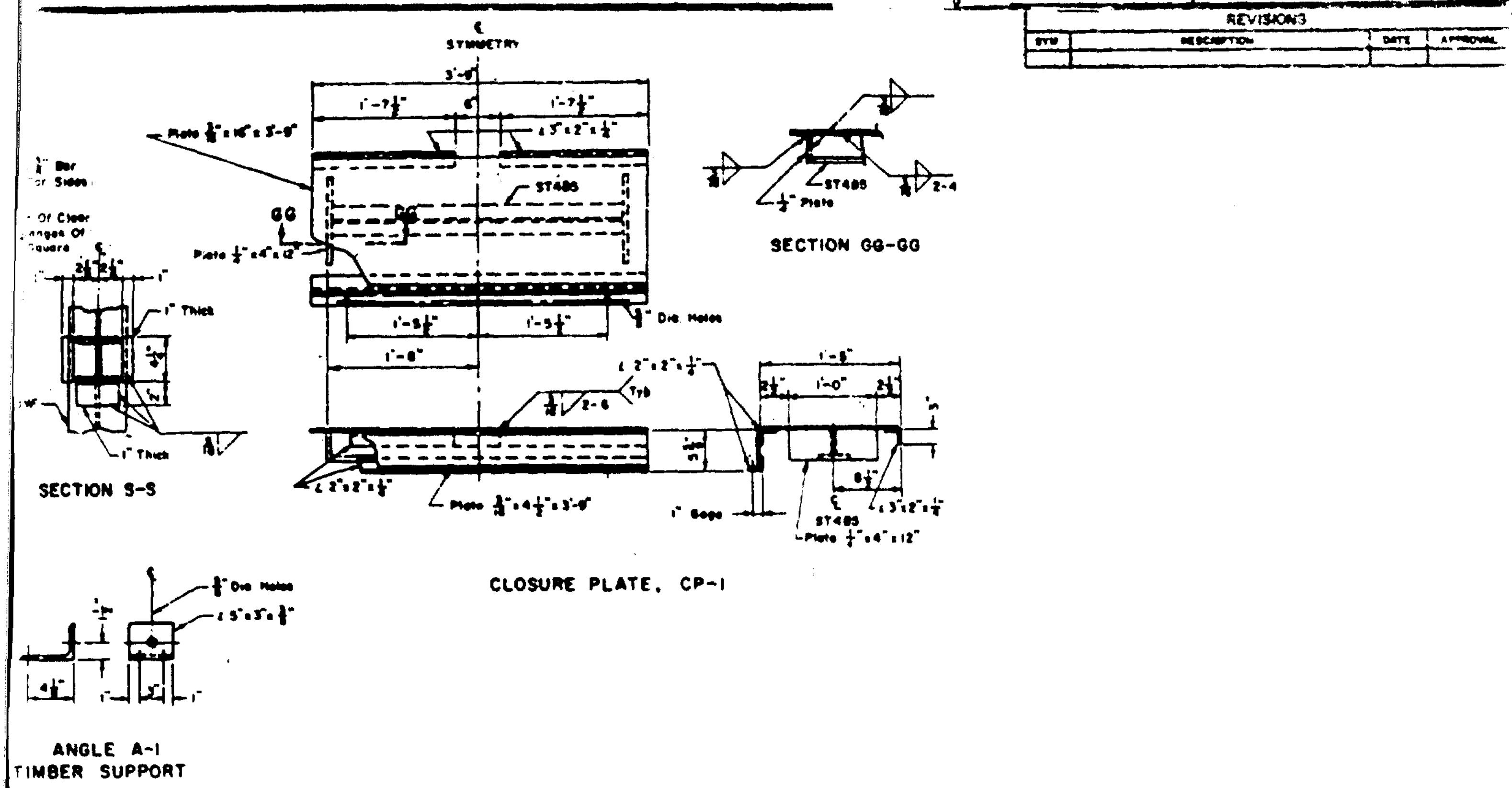


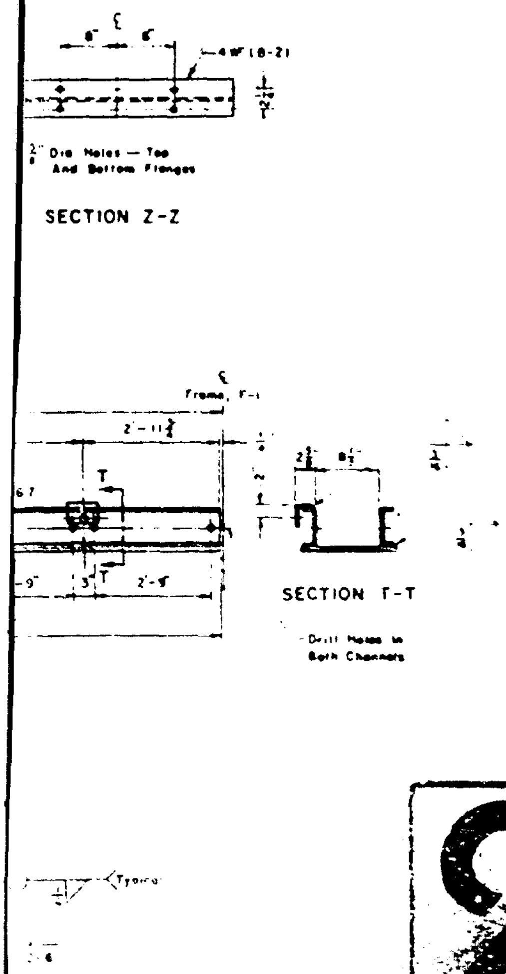
DETAIL "E"

2

APPROVED DIRECTOR	BULKHEAD TRUSS KIT 3	U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
RECOMMENDED DIV. CHIEF		CORPS OF ENGINEERS
SUBMITTED	PROTECTIVE SHELTER	VICKSBURG, MISS.
DESIGNED		N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-225
DRAWN / TRACED / CHECKED	SCALE / DATE 1 Sept. 1962	SHEET 5 OF 10


4 at 3'-0" = 12'-0"
CP-1
1/2" Bolts, 1 1/2" Long, 8 Req'd. Per Cover
A-1
PC-1
Entranceway Located Here
ERECTORS NOTE:
Position Of Entranceway Shown Is Typical. Locate Entranceway As Desired.
F-1
Frame, F-1
1/2" Dia Bolts, 4 1/2" Long, 4 Req'd.
16 W AT END OF SHELTER
2-5X6.7
S-1
Plate 12" x 1/2"
℄ Of End Truss
1/2" Bolts, 1 1/2" Long, 32 Req'd
B-1
SIDE ELEVATION
HH (See Sheet 7 of 10) HH
Stiffeners - Cut From 2 1/2" x 1/4" Bar And Locate On Near And Far Sides
8 W 20
Cut 4 W 13 1/8" Short Of Clear Distance Between Flanges Of 8 W's. Ends To Be Square
4 W 13
3/8" Dia Holes
Brace (B-2) To Be Inserted During Periods Of Alert
B-2
Note: All Joints To Be 1/4" Continuous Fillet Welds
8 W 20
3/8" Dia Holes - 3" Gage On Flange Of 8 W 20
3'-4"
END ELEVATION
PC-1 Not Shown
SECTION
PASSAGEWAY STRUCTURE - TIMBER OMITTED
3" x 12" x 12'-0" (T and G) (WS)
2" x 6" x 8'-0" (WSS)
(WSS) 2" x 6" x 8'-0"
3/4" Dia Hole (Typical), For 1/2" Dia Bolts, 3" Long
SIDE ELEVATION
3" x 4" x 5'-10" Nailer (BN) - Fasten To 8 W Web With 1/2" Dia Bolts, 4 1/2" Long
3" x 12" x 4'-0" (T and G)
(BS)
Two 3/4" Dia Holes In Each Nailer
Field Notch Timber At Bolt Heads
ERECTORS NOTE
When Utility Structure (Kit 5) Is Installed, Use End Bulkhead Shoring (BS) And Nailer (BN) From Kit 4 As End Bulkhead For Utility Structure. Nail BS To BN With 16d Nails
3'-4"
℄ 8 W 20
℄ 8 W 20
END ELEVATION
ANGLE
TIMBER S
SECTION
PASSAGEWAY STRUCTURE - TIMBER PLACEMENT
1'-4 1/2"
1'-0"
3/8" Dia Holes
ST4B5
2'-9"
2'-11 1/2"
PASSAGEWAY COVER, PC-1
4'-0"
Plate, 1/4" x 2'-11 1/2" x 4'-0"
∠4" x 3" x 3/8"
SECTION L-L
℄ Frame, F-1
4 at 3'-0" = 12'-0"
2'-11 1/4"
6'-0"
∠3" x 2" x 1/4"
3/8" Dia Holes
2-5X6.7
12" x 1/2" Plate
2'-9"
11'-11 1/2"
SILL, S-1
Lifting Lug, 1/2" Round Rod (Bend)
℄ Frame, F-1
3'-0"
2'-11 1/2"
2'-5"
∠4" x 3" x 3/8"
END VIEW, PC-1





GENERAL NOTES

1. Brace B-2 must be placed in position during alert periods.

2. The passageway structure (Kit 4) connects directly to the main shelter. The utility structure (Kit 5) does not contain bulkhead sheeting (BS), closure plate (CP-1) or extra frame (F-1). Consequently, the utility structure must be connected to a passageway structure.

3. If a vertical entranceway (Kit 6) or a ventilation structure (Kit 7) is installed above the passageway structure, the covers (PC-1) must be replaced by appropriate cover, PC-2, PC-3, or PC-4.

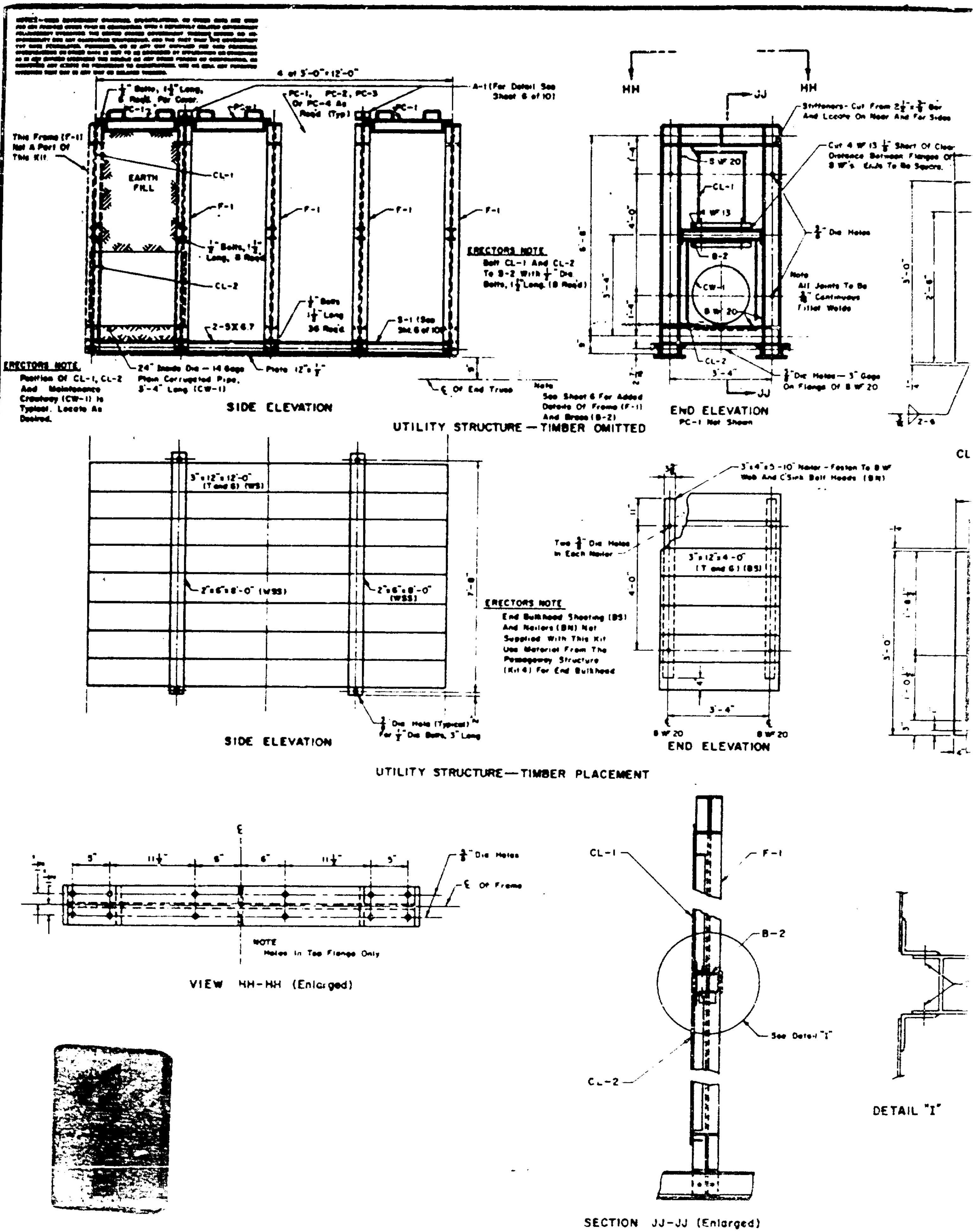
BILL OF MATERIALS KIT NO. 4 (PASSAGEWAY)

Material for ONE Kit No. 4 (1 req'd for 51 man shelter)

Piece Mark	Description	No. of Pieces	Unit of Measure	Total	Specifications
STRUCTURAL STEEL AND STEEL FASTENERS			lb.	lb.	
S-1	Sill	2	409	818	ASTM A-7 (Welded)
F-	Frame	5	475	2375	ASTM A-7 (Welded)
A-1	Timber Support	4	4	16	ASTM A-7
B-2	Brace	5	34	170	ASTM A-7
PC-1	Cover	4	220	880	ASTM A-7 (Welded)
CP-1	Closure Plate	1	110	110	ASTM A-7 (Welded)
	Hex Hd Mach. bolts and nuts 1/2" dia x 3"	8	0.25	2	ASTM A-307 Grade A
	Hex Hd Mach. bolts and nuts 1/2" dia x 1 1/2"	66	0.17	11	ASTM A-307 Grade A
	Bolts and Nuts 1/2" x 4 1/2"	8	0.33	3	ASTM A-307 Grade A
	Washers for 1/2" Bolts	164	0.04	7	
	Nails for Bulkhead, 16d			1	
	Subtotal, Steel and Fasteners			4395	
TIMBER			bm	bm	
WS	Wall Sheeting	16	36	516	See Sheet 1
WSS	Wall Sheeting Scab	4	8	32	
BN	Bulkhead Nailer	2	6	12	
BS	Bulkhead Sheeting	7	12	84	
	Subtotal, Timber		mbm	0.64	
			lb.	1490	
POLYETHYLENE SHEETING					
	One - 15' x 30' (Floor and outer surface of passageway)			22 lb	See Sheet 1
	One - 12' x 8' (end bulkhead)			5 lb	
	Subtotal, Polyethylene			27 lb	
	Weight/Kit			2.9 TONS	

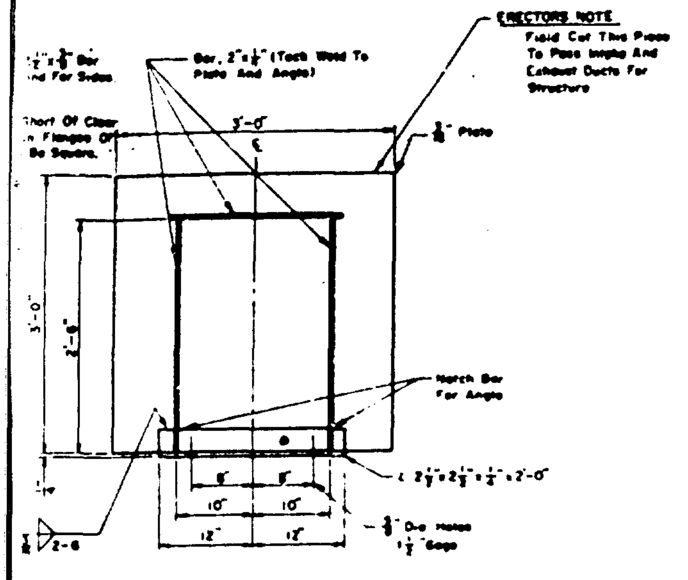
2

APPROVED: DIRECTOR RECOMMENDED: DIV CHIEF SUBMITTED: DESIGNER DRAWN / TRACED / CHECKED	PASSAGEWAY STRUCTURE KIT 4 PROTECTIVE SHELTER	U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS. N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-[illegible]
SCALE	DATE: 1 Sept. 1962	SHEET 6 OF [illegible]

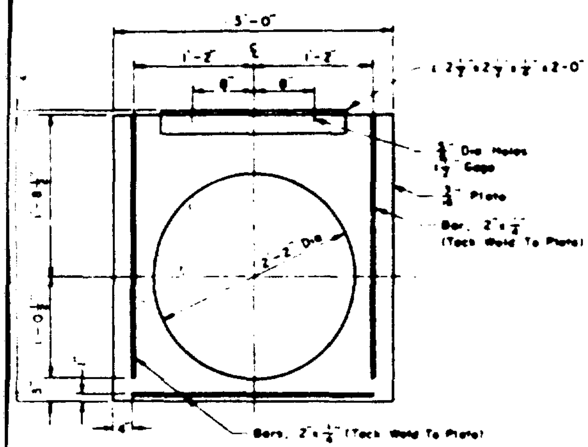


4 at 3'-0" = 12'-0"
A-1 (For Detail See Sheet 6 of 10)
PC-1, PC-2, PC-3 Or PC-4 As Req'd (Typ)
This Frame (F-1) Not A Part Of This Kit.
EARTH FILL
CL-1
F-1
CL-2
2-S X 6.7
S-1 (See Sht 6 of 10)
24" Inside Dia — 14 Gage Plain Corrugated Pipe, 3'-4" Long (CW-1)
Plate 12" x 1/2"
ERECTORS NOTE
Position Of CL-1, CL-2 And Maintenance Crossway (CW-1) Is Typical. Locate As Desired.
ERECTORS NOTE
Bolt CL-1 And CL-2 To B-2 With 1/2" Dia Bolts, 1 1/2" Long. (8 Req'd)
℄ Of End Truss
Note
See Sheet 6 For Added Details Of Frame (F-1) And Brace (B-2)
SIDE ELEVATION
HH
JJ
Stiffeners - Cut From 2 1/2" x 3/8" Bar And Locate On Near And Far Sides
Cut 4 W 13 1/8" Short Of Clear Distance Between Flanges Of 8 W's. Ends To Be Square.
8 W 20
4 W 13
B-2
CW-1
5/8" Dia Holes
Note
All Joints To Be 3/16" Continuous Fillet Welds
5/8" Dia Holes — 3" Gage On Flange Of 8 W 20
END ELEVATION
PC-1 Not Shown
UTILITY STRUCTURE — TIMBER OMITTED
3" x 12" x 12'-0" (T and G) (WS)
2" x 6" x 8'-0" (WSS)
7'-8"
3/4" Dia Hole (Typical) For 1/2" Dia Bolts, 3" Long
SIDE ELEVATION
3" x 4" x 5'-10" Nailer - Fasten To 8 W Web And C'Sink Bolt Heads (BN)
Two 5/8" Dia Holes In Each Nailer
3" x 12" x 4'-0" (T and G) (BS)
ERECTORS NOTE
End Bulkhead Sheeting (BS) And Nailers (BN) Not Supplied With This Kit. Use Material From The Passageway Structure (Kit 4) For End Bulkhead
3'-4"
8 W 20
END ELEVATION
UTILITY STRUCTURE — TIMBER PLACEMENT
5/8" Dia Holes
℄ Of Frame
NOTE
Holes In Top Flange Only
VIEW HH-HH (Enlarged)
CL-1
F-1
B-2
See Detail "I"
CL-2
SECTION JJ-JJ (Enlarged)
DETAIL "I"

REVISIONS			
SYM	DESCRIPTION	DATE	APP



CLOSURE PLATE, CL-1



CLOSURE PLATE, CL-2

BILL OF MATERIALS FOR KIT NO. 5 (UTILITY STRUCTURE)

Material for ONE Kit No. 5 (2 suggested for 51 man shelter)

Piece Mark	Description	No. of Pieces	Unit of Measure	Total	Specifications
STRUCTURAL STEEL AND STEEL FASTENERS			lb.	lb.	
S-1	Sill	2	409	818	ASTM A-7 (Welded)
F-1	Frame	4	475	1900	ASTM A-7 (Welded)
A-1	Timber Support	4	4	16	ASTM A-7
B-2	Brace	4	34	136	ASTM A-7
PC-1	Cover	4	220	880	ASTM A-7 (Welded)
CW-1	Maintenance Crawlway	1	84	84	ASTM A-245
CL-1	Closure Plates	2	89	178	ASTM A-7 (Welded)
CL-2	Closure Plates	2	66	132	ASTM A-7 (Welded)
	Hex Hd Mach Bolts and Nuts, 1/2" dia x 5"	8	0.25	2	ASTM A-307 Grade
	Hex Hd Mach Bolts and Nuts 1/2" dia x 1 1/2"	76	0.17	13	ASTM A-307 Grade
	Washers for 1/2" dia Bolts	168	0.04	7	
	Subtotal, Steel and Fasteners			4165 lb.	
TIMBER			fbm	fbm	
WS	Wall Sheeting	16	36	516	See Sheet 1
WSS	Wall Sheeting Scab	4	8	32	
	Subtotal, Timber		mbm	0.55	
			lb.	1250	
POLYETHYLENE SHEETING					
	One - 15' x 30', (floor and outer surface of passageway)			22 lb.	See Sheet 1
	Subtotal, Polyethylene			22 lb.	
			Weight/Kit	2.7 TONS	

ERECTION PROCEDURE (cont)

12. Construction of the passageway may proceed simultaneously with work at opposite end of shelter.

13. After vertical beams (B-1) have been fastened to arch ribs (AR-2), per Step 7, erection of the passageway may begin.

14. Prepare subgrade for passageway sills by placing 6" of well tamped backfill over the end truss. It is essential that the backfill be 6" above the flat leg of angles in the truss if frame (F-1) is to align properly with B-1.

15. Place the 30 ft. length of polyethylene sheeting such that the center of the sheeting is at centerline of passageway.

16. The 8' x 12' polyethylene sheeting in Kit 4 is to be placed at the closed end of the passageway structure (or the closed end of the utility structure if one is attached). The 8' dimension should be centered on the end of the passageway (or utility structure) with about 2' placed under the end of the sills.

17. Place passageway sills (S-1) in position approximately 2 1/2" from vertical beams (B-1). Small angles welded to sill must be on outside of passageway. Centerline of passageway to center on longitudinal centerline of shelter.

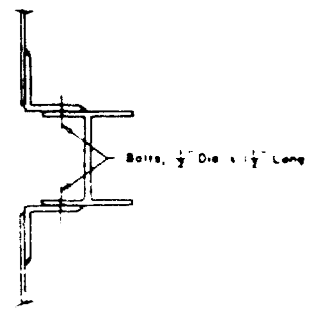
18. Insert ends of frames (F-1) in sill (S-1) adjacent to vertical beams (B-1). Connect F-1 to B-1 with four 1/2" dia x 4 1/2" bolts. Connect frame (F-1) to sills (S-1) with 1/2" dia x 1 1/2" bolts.

19. Place next adjacent frame (F-1) in position on sill (S-1) and loosely bolt with 1/2" dia x 1 1/2" bolts. Temporarily brace F-1 in position by using pieces of siding (WS). Place passageway cover (PC-1) in position bolting loosely to frames (F-1) with 1/2" dia x 1 1/2" bolts.

20. Continue erecting F-1s and PC-1s, as above, until all of the frames (F-1) in Kit 4, have been erected. One PC-1 should be placed without nuts on the bolts. See Step No. 24.

21. Place closure plate (CP-1) over top of beams (B-1) to cover spacing.

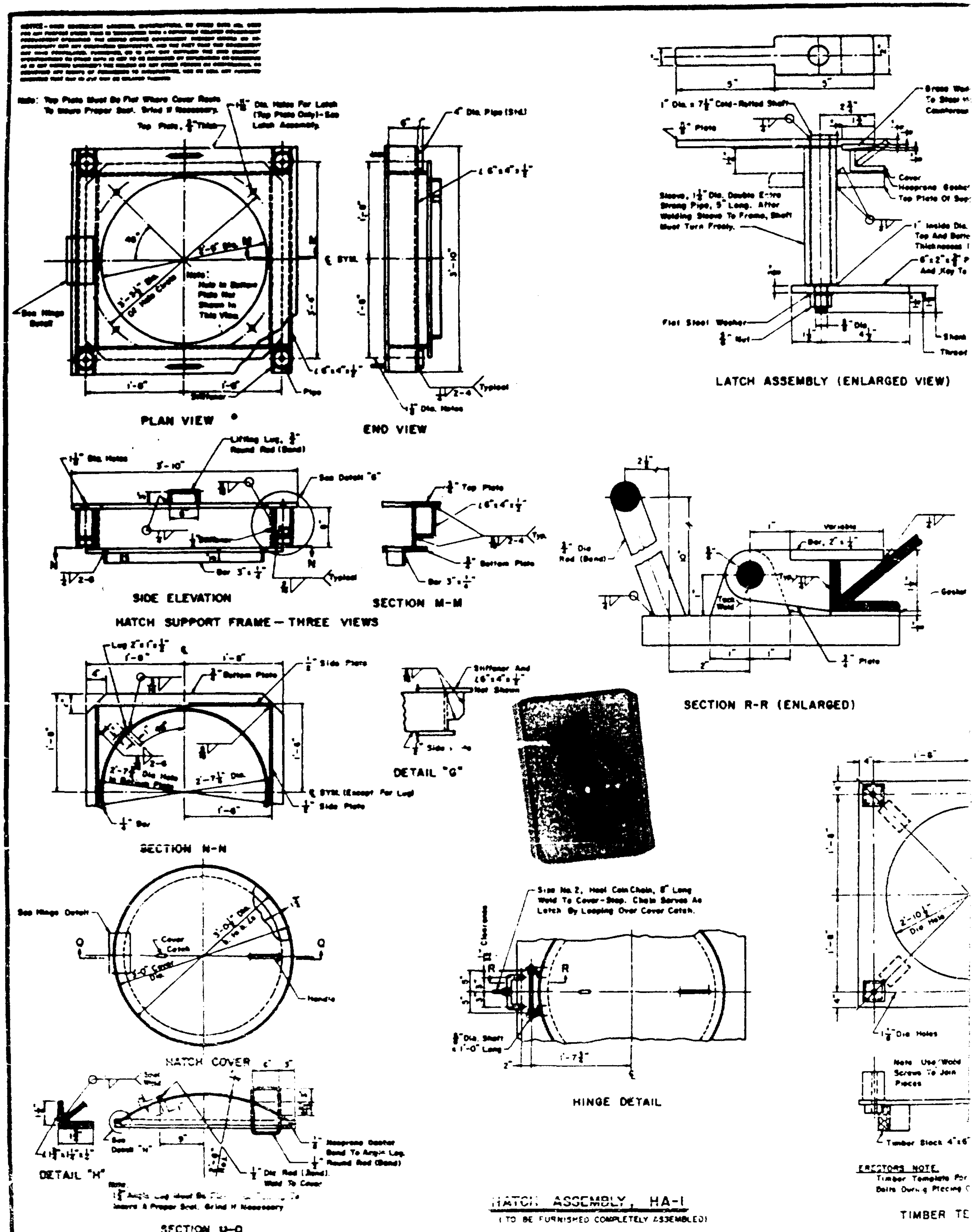
Continued on Sheet 8



DETAIL "Y"

2

APPROVED DIRECTOR RECOMMENDED DIV CHIEF SUBMITTED DESIGNER DRAWN / TRACED / CHECKED	UTILITY STRUCTURE KIT 5 PROTECTIVE SHELTER	U. S. ARMY ENGR WATERWAYS EXPERIMENT STA CORPS OF ENGIN VICKSBURG, MISS N. M. NEWMARK Urbana, Illinois Contract DA-22-079-e
	SCALE / DATE 1 Sept 1962	SHEET 7 OF



PLAN VIEW
END VIEW
LATCH ASSEMBLY (ENLARGED VIEW)
Sleeve, 1½" Dia. Double Extra Strong Pipe, 5" Long. After Welding Sleeve To Frame, Shaft Must Turn Freely.
Flat Steel Washer
SIDE ELEVATION
SECTION M-M
HATCH SUPPORT FRAME — THREE VIEWS
SECTION R-R (ENLARGED)
SECTION N-N
DETAIL "G"
HATCH COVER
HINGE DETAIL
Size No. 2, Heel Coil Chain, 8" Long Weld To Cover-Stop. Chain Serves As Latch By Looping Over Cover Catch.
DETAIL "H"
SECTION Q-Q
HATCH ASSEMBLY, HA-1
(TO BE FURNISHED COMPLETELY ASSEMBLED)
ERECTORS NOTE
TIMBER TE

REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL

Brass Wedge Attached To Steel-Handles By Countersunk Screws.

...rame Gasket
...late Of Support Frame

...Inside Dia. Brass Washers At ...op And Bottom Of Sleeve. Adjust ...hickness For Proper Latch Clearance.
6" x 2" x 3/8" Plate, Rd. Corners And Key To Shaft.

Shank Dia. Reduced To 3/8".
Thread Length.

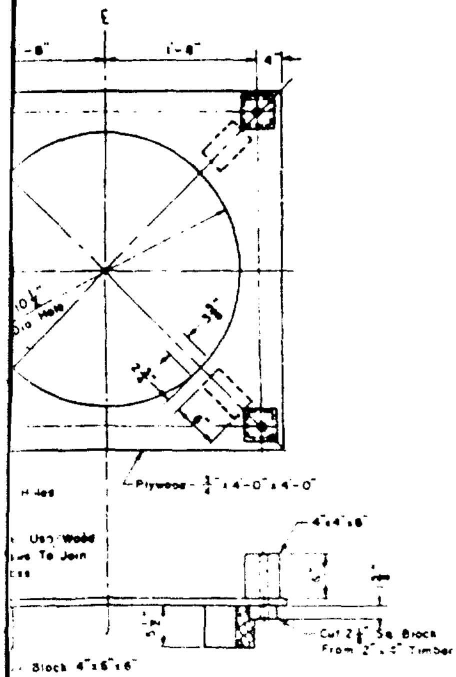
...VIEW)

— Gasket

BILL OF MATERIALS FOR KIT NO. 6 (VERTICAL ENTRANCEWAY)

Material for ONE Kit No. 6 (1 req'd for 51 man shelter)

Piece Mark	Description	No. of Pieces	Unit of Measure	Total	Specifications
STRUCTURAL STEEL			lb.	lb.	
PC-2	Cover	1	250	250	Complete Assembly, ASTM A-7
HA-1	Hatch Assembly	1	900	900	Complete Assembly, ASTM A-7
HSC-1	Supporting Columns	4	20	80	5" dia. Std. pipe
EP-1	Entrance Pipe	1	250	250	ASTM A-245
HTB-1	Tie Bolts	10	10.6	106	ASTM A-7
	Subtotal, Steel			1586 lb.	
TIMBER			bm.	bm	
EL-1	Entrance Ladder	1		(80 lb)	
TT-1	Timber Template	1	17	17	Complete Assembly
	Subtotal, Timber		bm	17	
			lb.	70	
CONCRETE		8	bag	800	Dry Pre-Mix Concrete - See Sheet No. 1
		weight/kit		1.2 TONS	



NOTE
Template For Positioning Of Pipes And ...ing Placing Of Earth Cover

...MBER TEMPLATE, TT-1

ERECTION PROCEDURE (cont)

22. All equipment including the crawlway pipe (CW-1) must be placed before the wall sheeting is erected on the utility structure. No instructions are included for the placing of any equipment. The closure plates, CP-1 and CP-2, are bolted in place above and below brace (B-2), respectively, with 1/2" dia x 1 1/2" bolts. As these closure plates form the bulkhead for the earth fill around the crawlway (CW-1) they must be placed such that the fill will hold them against their supports. Slide crawlway (CW-1) through the opening in CP-2.

23. The erection of the wall sheeting for the passageway may proceed simultaneously with the erection of the utility structure sills and frames. Loosely bolt wall sheeting seab (WSS) to clip angle on sill using 1/2" dia x 3" bolts. Place wall sheeting (WS) and bolt top of seab (WSS) to timber support clip angle (A-1). Tighten bolts in WSS.

24. If a vertical entranceway is to be installed in passageway, one passageway cover, (PC-1) must be replaced by PC-2 (from Kit 6). While this could be done at any time, it has been assumed that PC-2 will not be placed until the backfill has been placed along the sides of the passageway. For access to structure one PC-1 must be placed with the bolts in place but without nuts.

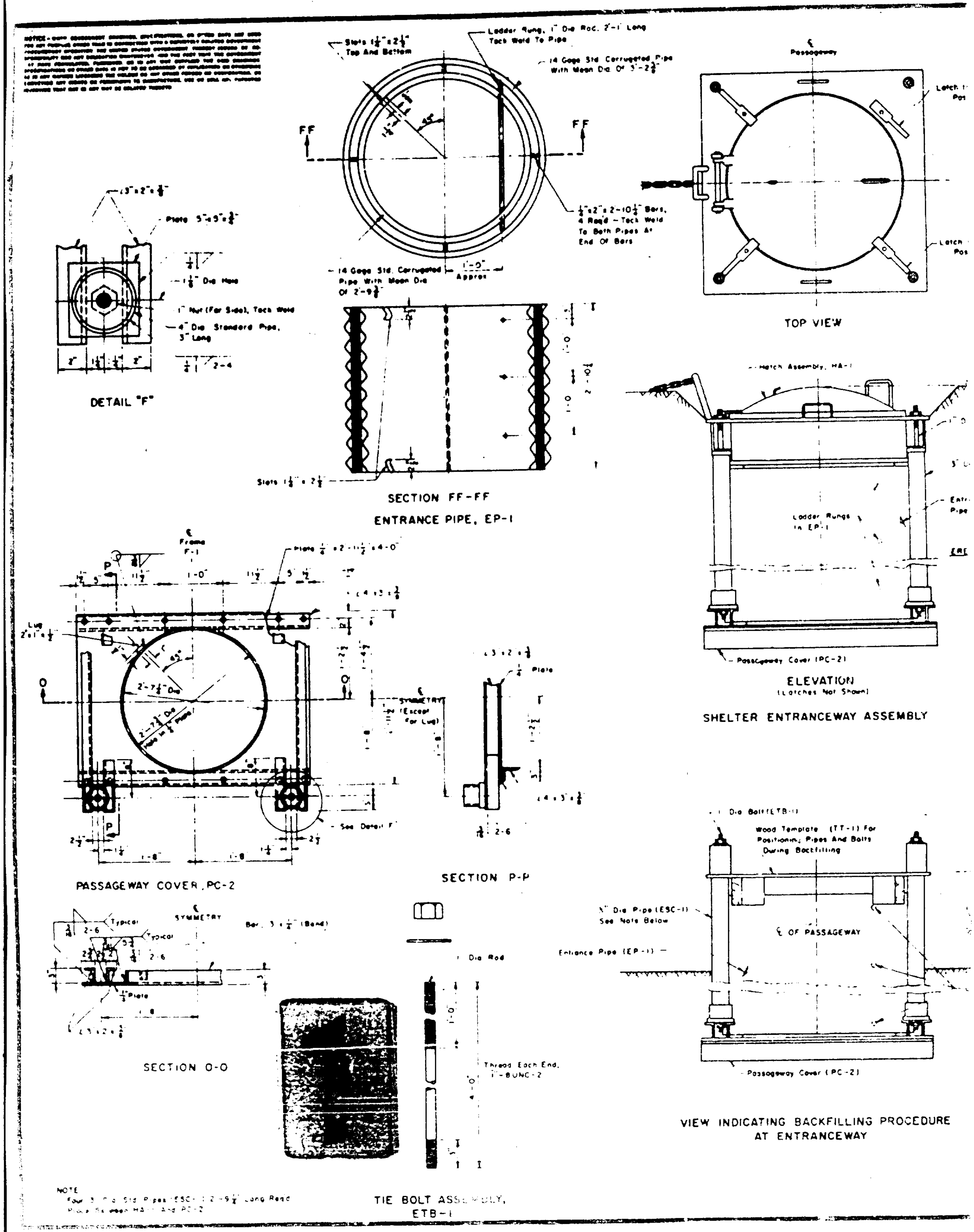
25. The end bulkhead sheeting, BS, is included in Kit 4; however, it must be placed at the end of the utility structure, Kit 5, if a utility structure is used. Bolt bulkhead nailer (BN) between the flanges of frame (F-1) using 1/2" dia x 4 1/2" bolts. Nail the bulkhead sheeting (BS) to BN using 164 nails. Field notch BS to clear bolts through BN.

26. Wrap the polyethylene sheeting around the sides and the end of the passageway and/or utility structure.

27. Backfill around the sides and ends of the passageway and utility structures in 6" lifts, hand tamped, up to, but not over, the roof of the passageway and utility structure.

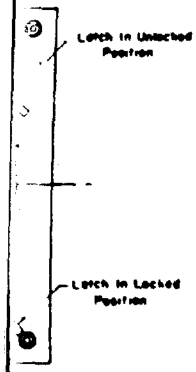
Continued on Sheet 9

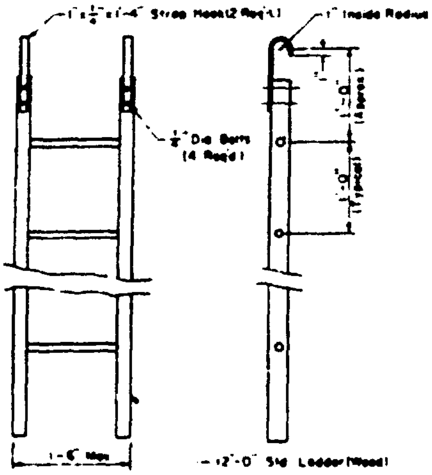
APPROVED / DIRECTOR / ... CHIEF / SUBMITTED / DRAWN / CHECKED	HATCH ASSEMBLY KIT 6	U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS.
	PROTECTIVE SHELTER	N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-...
	SCALE / DATE: 1 ... 1962	SHEET 8 OF 10



Slots 1 1/2" x 2 1/2" Top And Bottom
Ladder Rung, 1" Dia Rod, 2'-1" Long Tack Weld To Pipe
14 Gage Std Corrugated Pipe With Mean Dia Of 3'-2 3/8"
FF
1/2" x 2" x 2'-10 1/2" Bars, 4 Req'd - Tack Weld To Both Pipes At End Of Bars
14 Gage Std. Corrugated Pipe With Mean Dia Of 2'-9 3/8"
1'-0" Approx
SECTION FF-FF
ENTRANCE PIPE, EP-1
Slots 1 1/2" x 2 1/2"
Plate 5" x 5" x 3/8"
1 1/8" Dia Hole
1" Nut (Far Side), Tack Weld
4" Dia Standard Pipe, 3" Long
DETAIL "F"
℄ Passageway
TOP VIEW
Hatch Assembly, HA-1
Ladder Rungs In EP-1
Passageway Cover (PC-2)
ELEVATION
(Latches Not Shown)
SHELTER ENTRANCEWAY ASSEMBLY
℄ Frame F-1
Plate 1/4" x 2'-11 1/2" x 4'-0"
Lug 2 x 1" x 1/2"
2'-7 1/2" Dia
℄ SYMMETRY (Except For Lug)
See Detail F
PASSAGEWAY COVER, PC-2
1/4" Plate
SECTION P-P
℄ SYMMETRY
Typical
Bar, 3 x 1/2" (Bend)
1/2" Plate
SECTION O-O
1" Dia Rod
Thread Each End, 1"-8UNC-2
TIE BOLT ASSEMBLY, ETB-1
1" Dia Bolt (ETB-1)
Wood Template (TT-1) For Positioning Pipes And Bolts During Backfilling
3" Dia Pipe (ESC-1) See Note Below
℄ OF PASSAGEWAY
Entrance Pipe (EP-1)
Passageway Cover (PC-2)
VIEW INDICATING BACKFILLING PROCEDURE AT ENTRANCEWAY
NOTE

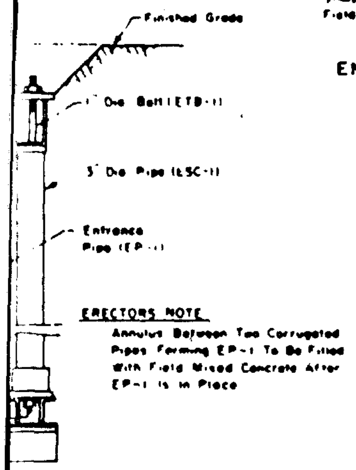
REVISIONS			
SYM	DESCRIPTION	DATE	APPROVAL





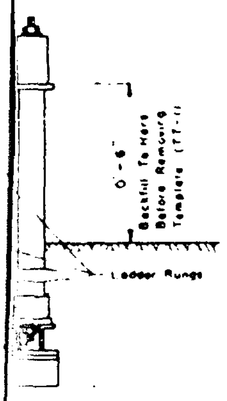
ERECTORS NOTE
Strap Hooks To Hook Over Lower Rung In EP-1
Field Cut Ladder To Required Length

ENTRANCE LADDER, EL-1



ERECTORS NOTE
Annulus Between Two Corrugated Pipes Forming EP-1 To Be Filled With Field Mixed Concrete After EP-1 Is In Place

BLY



EDURE

ERECTION PROCEDURE (cont)

28. Remove one or more covers (PC-1) to permit placement of backfill within the utility structure to form the earth bulkheads.

29. Remove and discard one passageway cover (PC-1) for each vertical entranceway. Install a passageway cover (PC-2) in its place.

30. Place entrance pipe (EP-1) in proper location over hole in PC-2. Note lug and slot for alignment.

31. Insert short threaded end of tie bolt (ETB-1) in hole in each corner of PC-2.

32. Place 5" dia. entrance support pipe (ESC-1) over each tie bolt.

33. Place timber template (TT-1) to center over entrance pipe (EP-1) and tighten the tie bolts securely to hold support pipes in proper position.

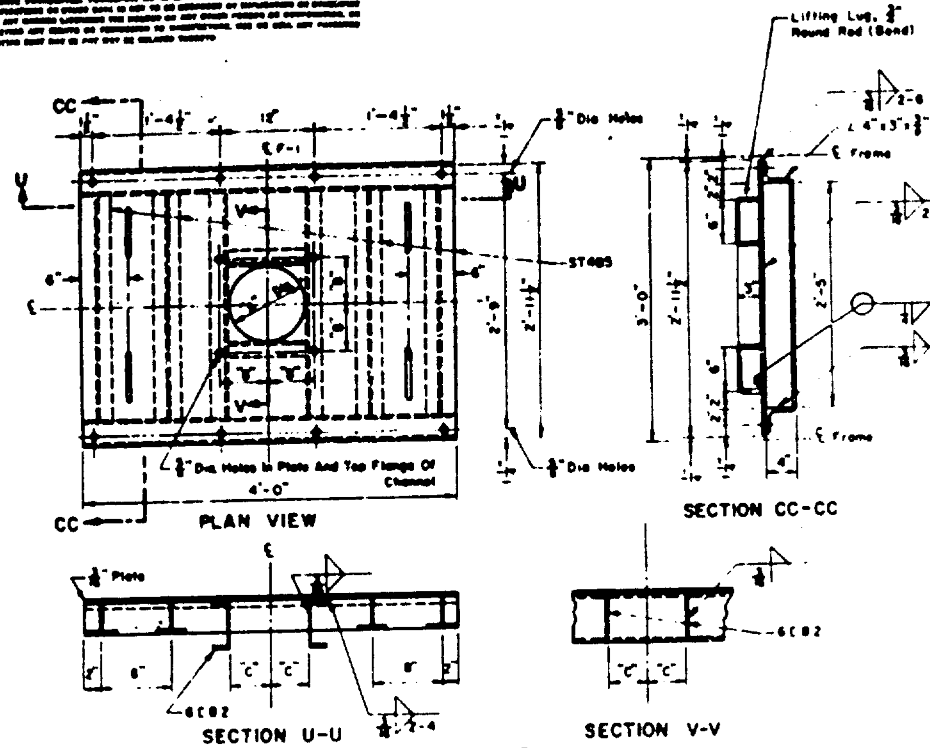
34. Backfill may now be placed around the entrance structure, as indicated on Sheet 9 to within 6" of the timber template, although it will be more desirable to backfill around entranceway and ventilation structures simultaneously. After backfilling to within 6" of timber template, the template should be removed and backfilling continued to the level of the entrance pipe (EP-1).

35. Fill void between corrugated pipes of EP-1 with field mixed, 5" slump, concrete. Rod thoroughly.

36. Place hatch assembly (HA-1) over the opening in (EP-1) threading the ends of the tie bolts through the corner holes in the hatch assembly. Tighten tie bolts securely and complete backfilling operations.

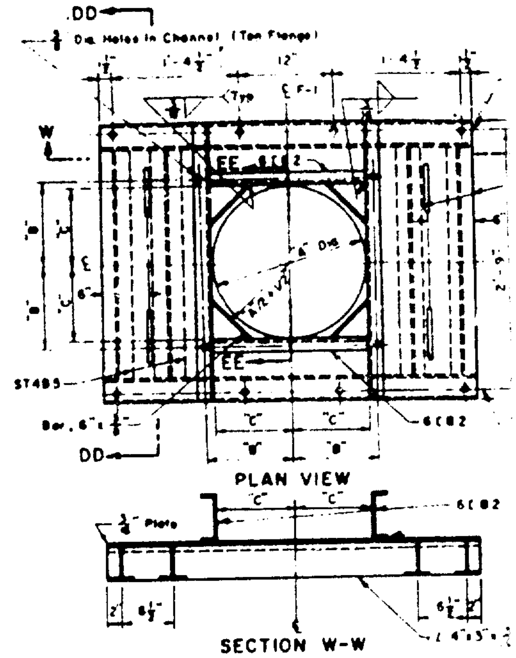
Continued on Sheet 10

APPROVED ... DIRECTOR RECOMMENDED DIV CHIEF SUBMITTED DESIGNER DRAWN / TRACED / CHECKED	ENTRANCE STRUCTURE KIT 6 PROTECTIVE SHELTER	U S ARMY ENGINEER WATERWAYS EXPERIMENT STATION CORPS OF ENGINEERS VICKSBURG, MISS N. M. NEWMARK Urbana, Illinois Contract DA-22-079-eng-22
	SCALE / DATE: 1 Sept. 1962	SHEET 9 OF 10



PASSAGEWAY COVER, PC-3

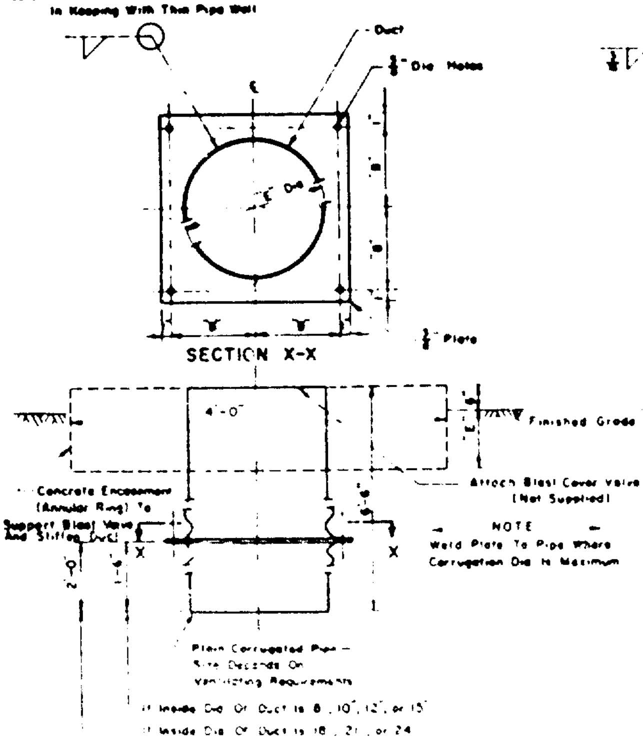
NOTE Use When Intake And Exhaust Ducts Have An I D Of 8", 10", 12", or 15"



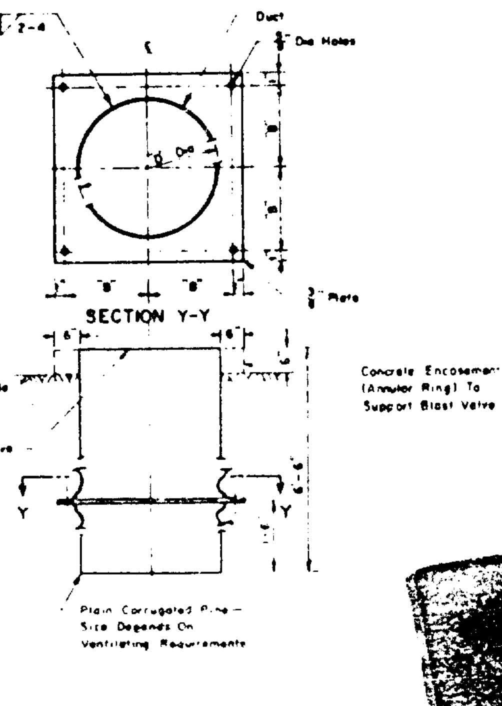
PASSAGEWAY COVER, PC-4

NOTE Use When Intake And E[xhaust Ducts] Have An I D Of 18" 21"

Note Weld Size To Be Maximum In Keeping With Thin Pipe Wall



AIR DUCT (CASE 2), AD-2



AIR DUCT (CASE 1), AD-1

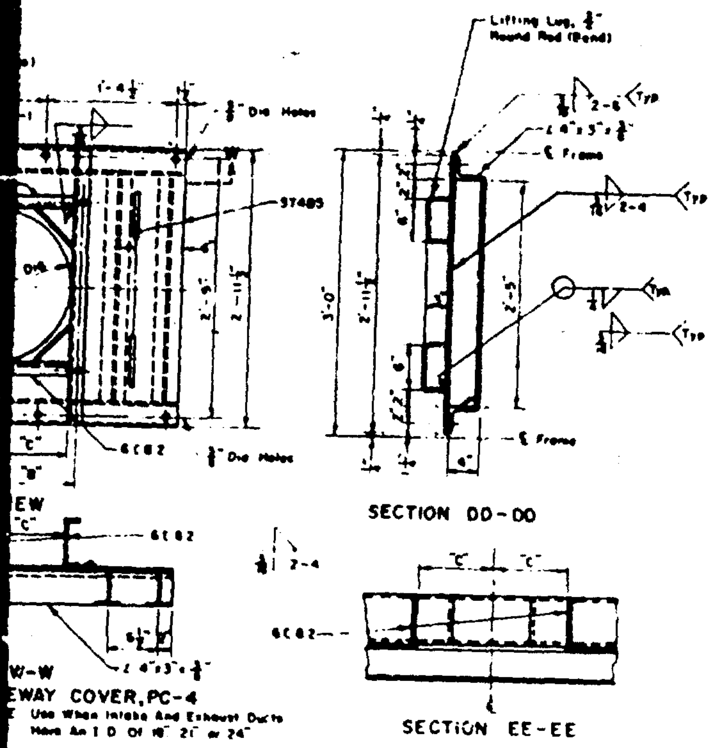


TABLE 1. [illegible]

Air Volume cfm	Air Duct I.D. in.	Wall Thickness Case 1	Wall Thickness Case 2	Pipe Dia Case 1	Pipe Dia Case 2
600- 900	8	12	16	10	10
900-1200	10	10	16	12	12
1200-2000	12	*	16	*	16 1/2
2000-3000	15	*	16	*	17
3000-5000	18	*	16	*	20
5000-6500	21	*	16	*	21
6500-9000	24	*	16	*	24

*No standard pipe will suffice; [illegible]
Case 1 - provides a ring of [illegible] to support the [illegible]
Case 2 - provides a large ring of [illegible]
valve.

TABLE 2. MATERIALS [illegible]

Air Volume cfm	Air Duct Weight lb.	Cover No.	Cover Weight lb	[illegible]
600- 900	80	PC 3	[illegible]	
900-1200	110	PC 3	[illegible]	
1200-2000	*	*	*	
2000-3000	*	*	*	
3000-5000	*	*	*	
5000-6500	*	*	*	
6500-9000	*	*	*	

Additional Materials:
4 - 1/2" dia. x 1 1/2" Hex. Head Bolts. Nuts [illegible]

Max. Weight/Kit = [illegible]

ERECTION PROCEDURE (cont)

37. The erection of the ventilation structures [illegible] vertical entrance structure, Step [illegible].

38. Remove and discard PC-1r at locations where [illegible] or PC-4, depending upon the required size of [illegible] this sheet).

39. Place air ducts (AD-1 or AD-2).
Backfill over the top of utility structure and [illegible]

40. Attach blast closure valves (Not supplied; [illegible]

41. Place concrete footing for blast closure valve [illegible]

2

APPROVED DIRECTOR	
RECOMMENDED DIV CHIEF	
SUBMITTED	
DESIGNER	
DRAWN / TRACED / CHECKED	SCALE